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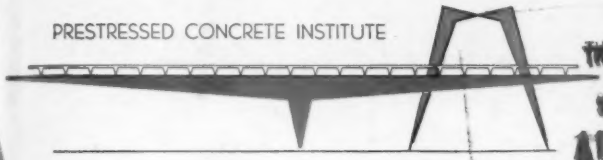

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March 1959 No. 4

JOURNAL

OF THE PRESTRESSED CONCRETE INSTITUTE

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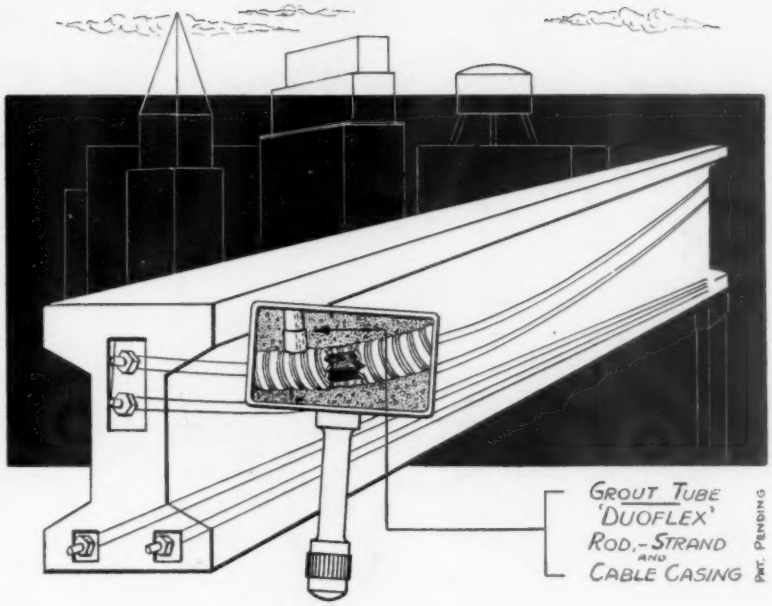

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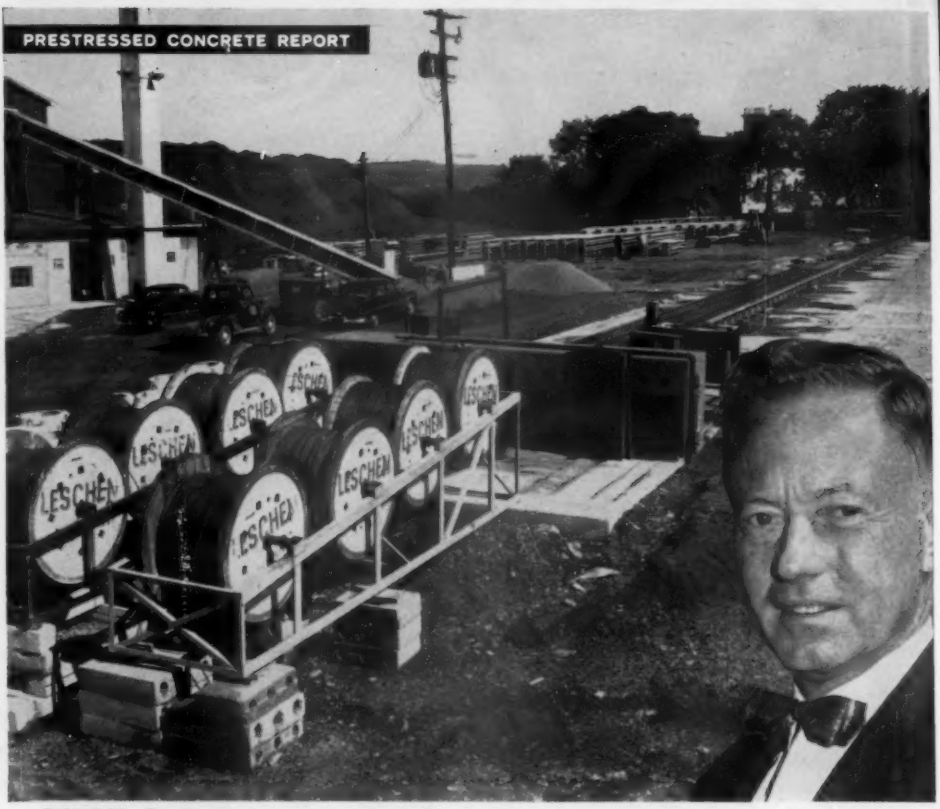

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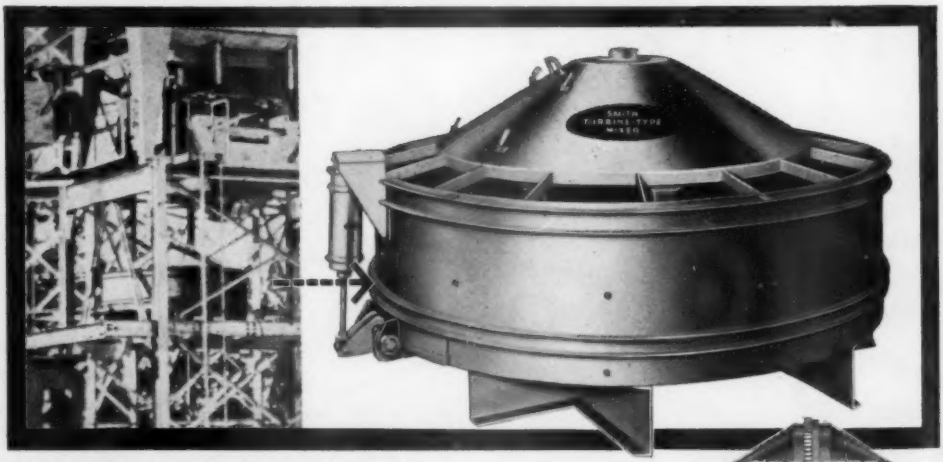

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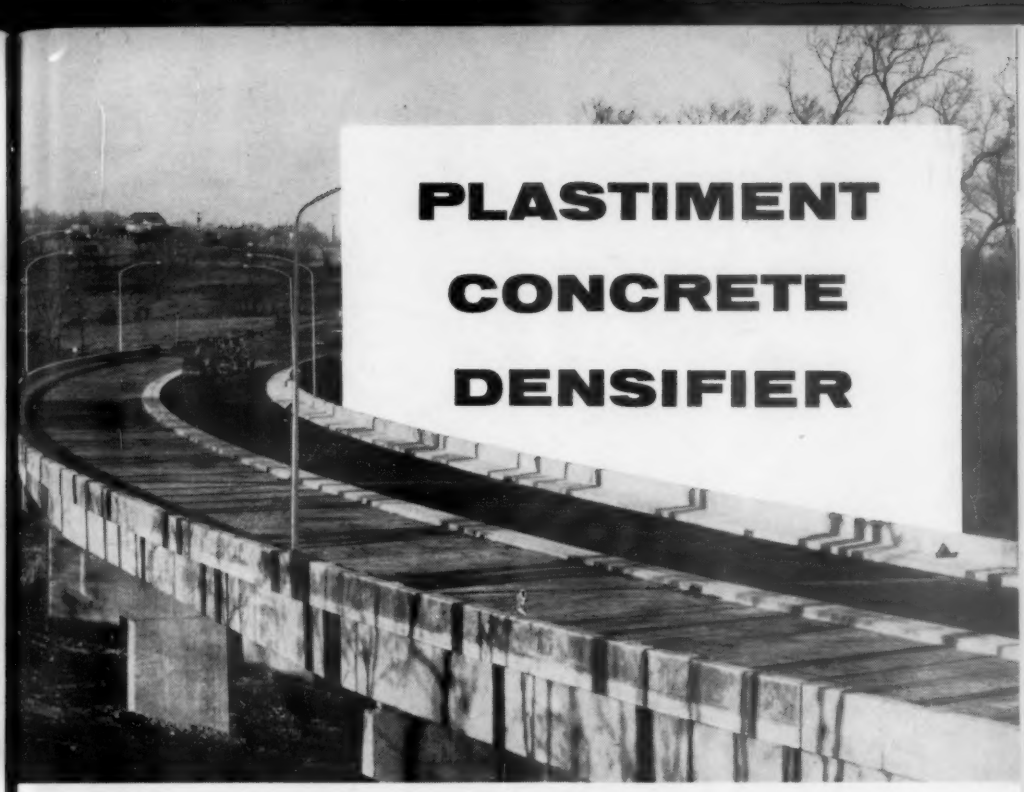

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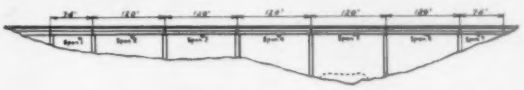

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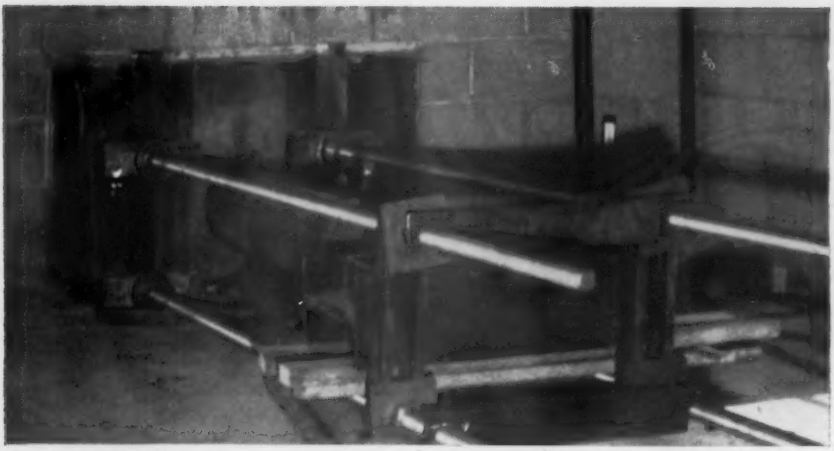

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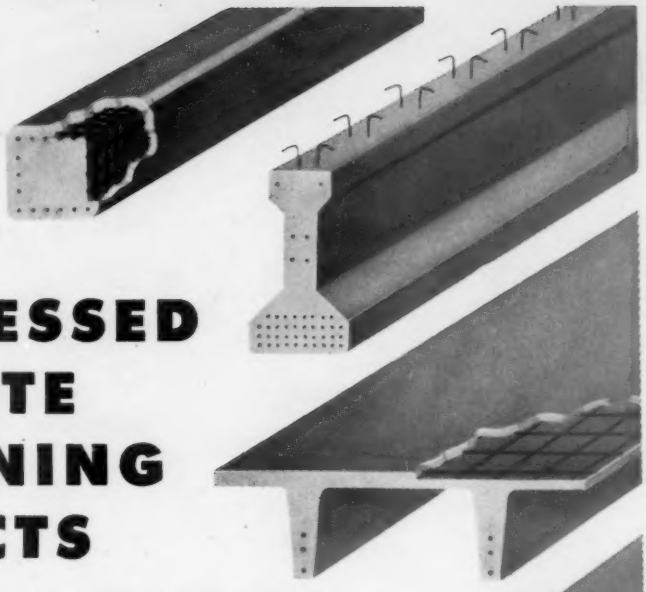

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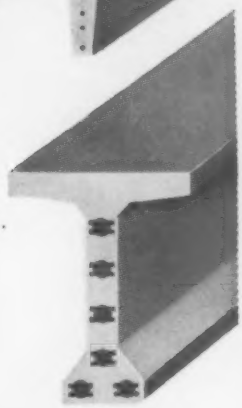

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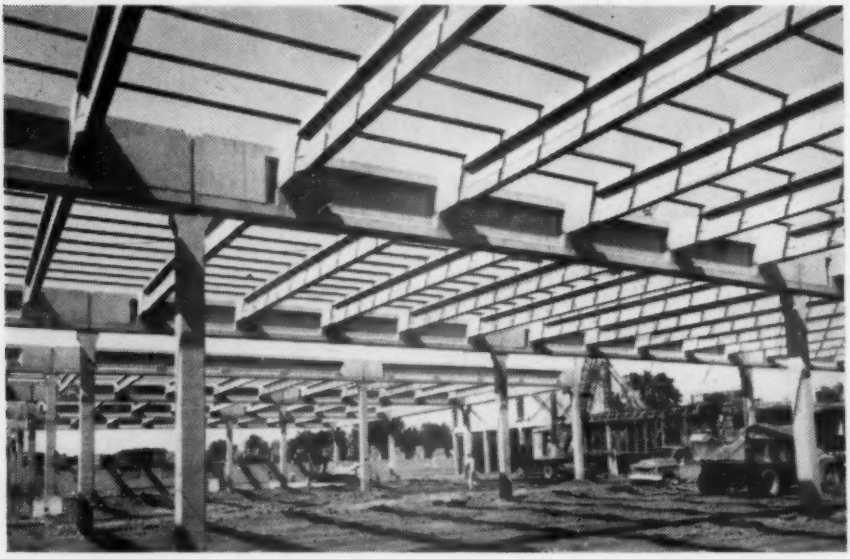

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LITERATURE EXCHANGE SCHEME

As a member of Federation Internationale de la Precontrainte, a Literature Exchange Scheme, agreed upon among all members, now makes it possible for us to bring to the JOURNAL readers the world of prestressing from all countries, wherever it is done. Articles of special interest will be selected on research, design, manufacturing and erection. Beginning with this issue, we are first bringing to our readers the General Reports of Sessions I, II and III, which are summaries of the papers presented at the Berlin Congress on Prestressed Concrete in 1958. You will note that each author's summary identifies by number each paper he presented.

If information on a specific subject is desired, it can be readily obtained from the "List of Papers" printed in this issue of the JOURNAL. You will thus have in capsule form the entire Berlin Proceedings, except for Session IV.

The total Proceedings of the Berlin Congress, with discussions of all papers, will be available at $40 prepaid, from Philip Gooding, General Secretary and Treasurer, Federation Internationale de la Precontrainte, Terminal House, Grosvenor Gardens, London S.W. 1, England.

—Peter J. Verna, Jr., President

Col. Martin P. Korn, Executive Secretary, will act as Editor pro tem. of the JOURNAL, pending the expansion of PCI Headquarters, and until such time as the Past Presidents' Committee present their program of development.

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What is P. C. I.?

By J. REX FARRIOR, JR.
Legal Counselor for P. C. I.
Tampa, Florida

P.C.I. is legally known as Prestressed Concrete Institute, Inc., a corporation not for profit organized and existing under the statutes of Florida, Chapter 617. The Institute is legally recognized as a scientific, charitable and benevolent association.

The original charter was approved by the Circuit Court at Tampa, Florida, on June 18, 1954. Officially, the object of the Institute is the formation of an association of manufacturers of prestressed concrete and of persons and companies in allied or related industries for the mutual exchange of information and ideas; and generally to do anything beneficial for its members and the prestressing business which does not violate the laws of Florida or the United States.

The charter provides for seven classes of members: Active, for prestressed manufacturers; Associate, for related businesses; Professional, for architects and engineers; Affiliate, for supervisory and technical employees, non-graduate; Junior, for architects and engineers in training; Student, for students enrolled in accredited architectural and engineering colleges, and Honorary, for such persons as the Board of Directors may wish to honor.

From a tax standpoint, the Institute enjoys an enviable position. Being a valid non-profit corporation, it is not required to pay any tax on its receipts, and therefore 100 per cent of its income can be devoted to research and other useful projects.

Fortunately, also each member's dues and contributions are completely deductible by the member for income tax purposes.

Both tax factors therefore guarantee to the Institute's members more than full value for each dollar invested.

QUALITY CONTROL FOR PRESTRESSING

A PANEL DISCUSSION

Presented at the 4th Annual PCI Convention

September, 1958, Chicago, Ill.

MODERATOR

Joe Schneider, Chief Engineer, Edward Campbell Co., Vineland, New Jersey

MEMBERS OF PANEL

Dan L. Sutter, Vice President, Anchor Concrete Products, Inc., Buffalo, New York

Robert Lyman, Chief Engineer, Atlas Structural Concrete, Inc., El Paso, Texas

Rudolph C. Valore, Jr., Director of Research, Texas Industries, Inc., Dallas, Texas

H. Colthurst, Technical Engineer, Ryan Builders Supplies Ltd., Windsor, Ontario, Canada

---

INTRODUCTION:

At the opening of the discussion, Mr. Schneider said that the discussions were to be held to questions of special interest to producers. He said that the questions and answers were completely unrehearsed and that while undoubtedly many divergent opinions would be expressed, they would be the genuine opinions of the panel members.

MODERATOR:

Mr. Colthurst, what do you have to say on the subject of quality concrete?

MR. COLTHURST:

Actually, as far as quality concrete is concerned, everybody will have a different mix design depending on the aggregates in their own territory. And then the main thing is uniformity. By having a quality control laboratory in your plant, your mix design can be checked thoroughly. Then quality is a matter of your curing and placing methods.

MODERATOR:

Mr. Lyman?

MR. LYMAN:

Basically, from a quality control standpoint, and from a quality concrete standpoint, our area in El Paso is a concrete masonry area and one of the big things in our area is the appearance of that concrete. I think probably along with compressive strength that appearance comes next in importance to the producer in El Paso, for the reason that the people there expect good appearing concrete. In fact, they demand it, and you don't sell your concrete unless it is good looking. So along with the other physical characteristics which everyone knows make good quality concrete, I think appearance is a prime factor.

MODERATOR:

Mr. Sutter, what are your comments on this subject:

MR. SUTTER:

Well, I'd first like to define quality concrete as I know it. I guess you've heard that quality concrete consists of a combination of aggregates, cement and water. And poor concrete consists of a mixture of water, cement, and aggregates. It's certainly the controls placed on it, and the people that exercise these controls, that make quality concrete. I agree with the two gentlemen preceding me, but I must say that in 16 years of producing high strength concrete that probably the most important step that we took to get quality concrete was the establishment of a laboratory. I certainly urge those that haven't taken this step yet to set aside something in the budget to develop a laboratory.

MODERATOR:

Mr. Valore?

MR. VALORE:

I agree with Mr. Lyman that appearance is very important. We find a very great amount of sales resistance on the matter of the appearance of double-T's or girders. Strength,

of course, we can determine. Strength has been a fetish, you might say, in the concrete field. Some believe that strength is the answer to all of the problems. However, there are other properties that are perhaps more important, that are not necessarily correlated with strength under all conditions. One of these is modulus of elasticity. Another very important property is drying shrinkage, and finally, creep or what was formerly known as plastic flow. It is possible to get good strength quickly and a fair modulus quickly and have a high drying shrinkage and creep. These poor properties can cause you difficulties after the member has gone to the job. Now it is possible to predetermine some of these properties. It is known, for example, that getting as large an amount of top size aggregate as is consistent with good placeability, is very important in producing low shrinkage and low creep. By selecting different aggregates for their mechanical properties, you can predetermine to a certain extent, modulus of elasticity. Strength is still the time-honored means of determining uniformity. We are firm believers in a fairly intensive degree of quality control and recognizing the statistical nature of concrete, that is, it is only statistically homogenous. We believe that statistical quality control, control charts, and so on, as outlined in the ACI Book of Standards, furnish a very good guide for quality concrete in plants. Unfortunately, there are some items that will resist even what seem to be fool-proof methods in producing quality concrete. There are the difficulties of weather in the various parts of the country; there are difficulties at times with non-uniformity in cement; such things as false set at times, and there are other problems that may appear after long periods of time.

MODERATOR:

Then I would say to summarize that basically what the panel feels goes to make up quality concrete includes a properly proportioned or designed mix, proper mixing after we design it, and a method for placing it to avoid segregation and of course care in curing. Some of these we are going to discuss in detail under other headings. In addition, I must agree wholeheartedly with Mr. Lyman that we must make good clean appearing concrete. I know in my own area the architect and owner are not interested in whether the concrete is 5,000 pounds or not. He's interested in whether or not his job looks good and quality concrete has a lot to do with how your product is going to look. And last but not least, there is adequate strength. Strength is definitely important. In fact it is a topic we are going to discuss in detail, but it is not the only criteria for quality concrete. Now to get into that a little further, we would like to discuss briefly what we are attempting to obtain and why with respect to strength in our various plants. First I would like to ask the panel this question: What usual strengths are attempted in your operations and what limits are you satisfied with in daily production? Mr. Colthurst?

MR. COLTHURST:

Our mix is all designed on a 6,000 pound strength.

MODERATOR:

This is a 28 day strength?

MR. COLTHURST:

This is a 28 day strength. Our pouring cycle is 16 to 18 hours and our strength at release is between 35- and 3600 psi to above 4200 psi. We never lift a plank under about 3500 to 3600.

MODERATOR:

And you say your limits do vary between about 3600 to 4100 psi? How about your operation, Mr. Lyman, in El Paso?

MR. LYMAN:

We have a two-sided operation going at the same time because the Texas highway department requires strength of 4,000 psi at tension release and we pretty well, on our double-T operation also, try to shoot for the 4,000 psi. But depending on the particular application and depending a little on just how the panel is going to be used, and where it is to be used, we go as low as 3500 psi, but not below. Most of the time from a release standpoint, our strength on double-T's, I would say, runs from 3800 to 3900 psi and up to 4600 psi. Of course on beams, it has to be over 4,000 psi.

MODERATOR:

Mr. Valore, what do you strive for in your operation in so far as release strengths are concerned?

MR. VALORE:

Well, we too are under the jurisdiction of the Texas highway department and the 5,000 psi requirement at 28 days is of no particular concern to us, as the speaker in the previous panel indicated. Twenty-eight day strength is largely an academic matter, and, besides, we're much more interested in getting the 4,000 psi required for the tensioning as early as possible, generally speaking in the order of 18 to 20 hours. When we are able to do that using type 3 cement and steam and again with steam we are limited to a maximum of 140 degrees by the Texas highway department, we find that for such concretes the 28 day strength water cured, will be on the order of 6,000 to 7,000 or 8,000 psi and occasionally

as high as 9,000 psi. We also make lightweight aggregate, double-T's and have made some lightweight aggregate post-tensioned girders. And there we attempt to get 3,500 to 4,000 psi before stressing. In our minds again we are much less concerned with strength and more concerned with what we think are the more important properties like, say, the modulus of elasticity at the time of stressing or pretensioning.

MODERATOR:

For operation in New Jersey we strive in our double-T operations for a 4,000 release strength in 24 hours, primarily because that is the release strength used by the highway department in the bridge beam operation. There is nothing wrong, I feel, in release at lower strengths if the panel, the double-T panel, is not over-stressed or over-cambered. In my mind the important thing is to get uniformity of release strength day by day in our operations. Our daily strengths vary only between 4,000 and 4,300 and that's about it. And I think we'll discuss a little later why we feel that this uniformity of release strength is so important. The next question that we were going to discuss was the usual strengths attempted by this panel at 28 days. I think we agree on that one that it's a rather academic question. If you get your release strength in 24 hours or in whatever time you want to get your release strength, then the 28 day strength normally takes care of itself. However, what is the usual 28 day strength specified in your area?

(Members of the panel reported specified 28-day strengths in their areas, ranging from 5,000 to 6,000 psi. Mr. Lyman reported that when they obtained release strength of 4,000 psi, they achieved 28-day strengths of between 6,900 and 7,200.)

MODERATOR:

Mr. Valore?

MR. VALORE:

Well, we are operating under the same requirements as Mr. Lyman. We are not quite as fortunate in maintaining our strengths within as narrow a range as 6900 to 7200. But generally speaking, we have no difficulty whatsoever in meeting 5,000 psi, whether it's in lightweights, sand and gravel, or limestone aggregate concrete at 28 days. One of the reasons perhaps is that we use high early strength cement entirely for our precasting operations.

MODERATOR:

The next question is how quality concrete affects camber, and I would like to open the discussion on this particular question because it has been very close to my heart and I've given an awful lot of thought to it. Frequently, I have many visitors come to me from all over the country to look at our operation, and the usual greeting is, "Hello, I'm Mr. X. How's your camber?" Camber, I think, is a much ado about nothing subject in the pre-stressing field. And I'm talking now particularly with reference to roof panels. I think that the problem that the industry is faced with is not camber but differential camber. The architect usually isn't too concerned about the bow he has in his roof, it's the difference in bow between panels that's got him worried. And, basically, there are only two things which affect differential camber and we have control of both. One is the amount of pre-stressing force, and, two, is the concrete. Now, the first is actually something that is practically automatically controlled. We have a given number of wires for a particular span and load. We have a given eccentricity. We usually deflect. If we deflect the wires in the same position day after day, and it just takes a visual check so that this is done properly. We all jack, and have two methods of controlling that, the gauge and the elongation measurement, and, certainly, this is no difficult task for the producer. The next thing is the concrete — and herein lies the big problem insofar as our differential camber question is involved. All too often, I feel, that the producer is not paying enough attention to producing quality concrete and he can eliminate a lot of his camber problems by doing so. YOU MUST get a high strength concrete and the same strength concrete from day to day, and you have to do that either by producing the same kind of concrete from day to day, or varying your release time so that you are releasing every day at the same strength. Obviously, the most economical thing to do is to make the proper concrete and release at the same time every day, so that you won't be shifting your production cycle. In my plant, we have very little trouble with camber. In fact, I have erected many times, 8,000 to 10,000 square feet of double-T's. 50-foot span, without having to jack any of them to kill differential camber. The maximum differential camber on my 50-foot double-T's is a half inch. The total camber usually is unimportant. In fact, the customer comes to my plant and compliments me on building the drainage into his roof for him. Now, Mr. Colthurst, how do you feel about this subject?

MR. COLTHURST:

I agree with you 100 per cent. The important thing is to perform your whole pouring and curing cycle on a uniform basis all the way through. You'll have very very little trouble with camber control if your mix is the same, you pour at the same time, and you release at the same time.

MODERATOR:

Mr. Lyman, do you have anything to add to that?

MR. LYMAN:

Well, I'm going to be different. I'm only going to agree 50 per cent because of one fact. I think basically the agreement is that in roof structures, camber really doesn't make any difference most of the time. But then, when you get into a floor system, you have a horse of another color. Anyway, we have this problem and it is simply a matter of education for the architects. When they design a floor system, we estimate for the architects, in advance of production, approximately what his camber is going to be, so that if he feels he has to have a level, a dead level floor, he then can lower the ends of the panel so that he gets more topping at the end of the panel than he does at the center. Sometimes we go along with a little bit less than the usual 2-inch topping but don't like to go under an inch and a half topping at the center of the panel.

MODERATOR:

Mr. Sutter?

MR. SUTTER:

Well, I was interested in your remarks regarding uniformity of mix. Certainly this is an important point in this particular question. I would like to touch on one point that was on the agenda of the first question, and that's a matter of segregation. Certainly segregation is not a good thing to have if you want uniformity. One of the things that isn't thought of too often is the method of transportation of your mix to your mold and segregation can occur at that time. Another thing, is the material that you vibrate off the top of the mold. Now this is probably your poorest concrete, it's full of water and you wouldn't want a large portion of it in the middle of your mold.

MR. VALORE:

I can only second what the other speakers have said about the desirability of uniformity of camber.

MODERATOR:

I must agree with Mr. Lyman that insofar as floor slabs are concerned, you're going to have camber in your floor slabs especially, because there you are using more prestressing force on shorter spans. When comparing them with a roof panel you are going to get more camber. That is purely a matter of education of your architect and your contractor before they put the bid in. Tell them what camber to expect. Tell them how much concrete deck they are going to have to cover and how much topping they are going to have to estimate and place on the job, and you won't have too much of a problem in selling prestressed double-T's for floor decks. Our next question I would like to address to Mr. Lyman. How do you obtain quality concrete, and how do others down your way?

MR. LYMAN:

One of the major problems which confronts any prestresser is the continual running battle with nature and the atmosphere. It seems that a good share of the time, the prestressed concrete producer is concerned with concreting operations under severe weather conditions. For this portion of the discussion, let us examine some of the difficulties involved in hot weather concreting and try to find some solutions to these difficulties. One of the most critical areas in the United States where all these difficulties seem to crop up is in the hot, arid, desert-like region of the United States, including Texas, Arizona, New Mexico, Utah, Nevada, and California. Much of this area experiences many summer days when the temperature is in the 90 degrees to 110 degrees Fahrenheit range, accompanied by low relative humidity in the vicinity of 2 per cent to 10 per cent coinciding with wind velocities upwards of 15 to 25 miles per hour. Unless proper precautions can be taken this set of conditions will tend to affect several different properties of the concrete. First, the ultimate strength of the concrete may be impaired because the temperature of the concrete as deposited in the forms is too high. Experience of the Bureau of Reclamation in this area, shows that concrete temperature at placement should not exceed 90 degrees Fahrenheit, and even this is too high. The preferred temperature range is 70 to 80 degrees. Cracking tendencies are increased because of the greater range between the high temperature at the time of hardening and the low temperature to which the concrete will later drop. Unit water requirement for mixing is increased by higher concrete temperatures which contribute to greater drying shrinkage. Also, concrete that is mixed, placed, and cured, at high temperatures has been found to fail sooner than concrete mixed, placed and cured at lower temperatures. Some measures which may be employed to lower the temperature of concrete as mixed are: (1) using cool, or cold mixing water, (2) avoiding insofar as practicable the use of hot cement, (3) cooling coarse aggregate with water by sprinkling or inundation, (4) insulating mixer drums or cooling them with sprays, or wet burlap coverings, or shading materials. Still another serious difficulty produced by the weather conditions is the difficulty of securing continuous moisture for the required period. Although attention to the curing requirements is important at all times, it is especially so in hot, dry weather because of the greater danger of crazing and cracking. Higher temperatures with low humidity cause sprinkled surfaces to dry faster and to require more frequent sprinkling, hence, the use of wet burlap or blanketing and other means of retaining the moisture for longer periods becomes increasingly desirable. Efficiency of curing compounds is reduced in hot weather. In our plant in El Paso this has proven especially true. In fact, we have eliminated the use of curing compounds in connection with

the top surface of the double-T's during the hot summer months. We use curing blankets entirely and these curing blankets, I might add, are the standard variety as specified by the Texas Highway Department. We usually apply these blankets immediately after the completion of our screeding and finishing operation. Wet blanketing in contact with the concrete is excellent for this purpose. It not only shades the concrete, but also holds the moisture needed for good, moist curing. During the planning stages of our casting bed in El Paso, I considered at length the various means available to promote rapid curing, to obtain higher strength concrete essential to our prestressed concrete operation. The final decision on the curing means was in favor of steam curing and a factor affecting this decision was the advantage that would accrue from the moisture gain that is inherent with saturated steam. Our steam curing has proved to be a distinct benefit in overcoming our low relative humidity as well as providing heat to obtain the high early strength required. I don't mean to propose that steam curing is the only way to do it because there are other means that have been used successfully in other areas, such as hot oil and hot water and even radiation, but I think in our particular case, steam curing was the best answer. My previous remarks were made using the desert area of the southwestern part of the United States as an example because I believe this area presents the most problems to plant concreting operations in hot weather due to the prevalence of hot sun, high temperature, low humidity, and windy conditions, and because these conditions can occur concurrently. If you eliminate the factor of low humidity you approximate the worst condition that occurs occasionally in the remainder of the United States, and of course, the same precautions mentioned are appropriate and can be applied.

MODERATOR:

Thank you, Mr. Lyman. All I can say is that I'm glad I'm not pouring concrete in Texas. Now we would like to jump from Texas up to Canada and Mr. Colthurst. How do you handle concreting in cold weather?

MR. COLTHURST:

Let me assure you that we have no trouble in obtaining quality concrete in cold weather, in the plant. You all have the same literature available that I have. So to cover this subject I think I'll explain what we do and hope that this may be of some assistance to you. We started prestressing on our own in 1953 and then in 1954 we affiliated with Leap Concrete. In that year we set up two 300-ft. double-T beds, one at Windsor and the other at London. As we were one of the first producers in the north, we did not have the benefit of the previous experience of others, so had to proceed on our own. Our beds are solid concrete with steel stem liners only. And although these do not have some of the advantages of the newer all-steel forms, they are, I think, more stable and better in colder climates. Our main problem, of course, was supplying heat uniformly to the bed. The most logical course was to supply heat to the center of the bed and branches to both ends. So from a central header 2-inch cast iron pipe was threaded through a 6-inch rain water leader and preset to the correct fall, approximately at the mid-point of the stem of a 14-in. double-T. We looped the pipe at the ends for expansion and drainage purposes. This has proved entirely satisfactory during the last four years, enabling us to place our concrete at 85 to 90 degrees and then rising, after a period of two to three hours, to 140 degrees. The heating is all steam from a 30 HP automatic, oil-fired boiler which we use all through the year, both summer and winter. We have our own ready-mix plant and place concrete with ready-mix trucks. In the winter we use heated aggregates. On the edge of the bed we ran a ¾-inch water line which, in the summer months, we use to spray, spraying immediately after pouring. In cold weather we pipe steam into this line. Strangely enough, we have less trouble in the winter than we do in the summer. And, although our bed is entirely outside, we've lost only a few days due to bad weather conditions. We've never used any admixtures.

MODERATOR:

The next question that we would like to discuss is probably one of the most discussed topics in our industry, and that is curing of concrete. I would like to have each panel member state briefly but in detail, how they cure in their various plants. I think you've covered that, Mr. Colthurst and Mr. Lyman. How about winter time, do you make any changes in the winter other than what you discussed under hot weather concreting?

MR. LYMAN:

About the only thing we do differently in winter is to use a membrane curing compound on top of the double-T's.

MODERATOR:

Mr. Sutter, what wrinkles do you use in Buffalo for curing concrete?

MR. SUTTER:

We cure with moist steam. But one of the things that we added that has helped us produce quality concrete is a secondary curing kiln. Our primary curing is at about 130 degrees. The secondary kiln is set at 100 degrees and it has the advantage in the winter, of bringing temperature up slowly. We are not blessed with hot weather difficulties in Buffalo, we are certainly up against it with the sub-zero temperatures. We don't believe that a precast member should be taken out when it is too hot to handle and put right out in zero weather.

MODERATOR:

The next question is how do we get control of our concrete in daily production in our plants? Mr. Sutter again, what steps do you take to obtain quality control in the production of concrete in your plants and what control facilities do you have available both in terms of equipment and men?

MR. SUTTER:

I predicate my remarks with the fact that we do have a laboratory. Quality control is governed by a laboratory man. He not only has responsibility, but this man has authority which is important. As far as the aggregates are concerned, we have a sieve analysis done about once a week. Of course we heat our water and aggregates both at the bin and at the hopper. We heat our water during cold weather. We have a paddle-type mixer with a Teleslump meter on it. We haven't been depending entirely on that for our slump so we make slump tests three or four times per shift. As to the cement we expect to get a Blaine fineness meter. We take 9 cylinders a day in our laboratory and break them in sets of three, the time intervals depending on what we want them for, stripping, 28-day strength or what. Of course the time and temperature charts on our kilns are important.

MODERATOR:

Mr. Valore, what steps do you take at Texas Industries to achieve actual quality control in your plant production?

MR. VALORE:

We have a full-time man assigned to the job of quality control in our structural division. The laboratory is part of the parent company and is not a part of company making the prestressed members. In our lightweight work we make hourly checks: Of each two cylinders one will be held for 28 days, and the other one will be broken at 24 hours. We have determined over a period of time the relationship between 24-hour strength and 28-day strength, depending on the type of curing. We use high early strength cement in all instances. As most of you know who have worked with lightweight, the problem of control is considerably more difficult than it is with sand and gravel and limestone sand mixes. Also weight enters into your control of the situation very definitely, particularly with respect to yields and determination of cement factor. With regard to our work for the Highway Department, our control is pretty much limited by the designs and procedures set up by the department and there is no departure from those requirements. Generally speaking, for a bed 440 to 460-ft. long, there would be 9 or 10 beams or girders. For pretensioning we are rquired to show an average of 4,000 psi for 3 cylinders. The Highway Department does not specify how those cylinders shall be taken, that is, how many batches shall be required. But for our own safety we have decided to provide cylinders from three separate samples of concrete. The end of the pouring of a bed is perhaps the more important. We take the samples near the beginning of the pour, in the middle and near the end. Two cylinders are taken from each sample so that if the first one doesn't make it, we're left with another that can be tested some hours later and again come out with the average of three cylinders at 4,000 psi. Any cylinders left over are held and become our 28-day cylinders. Curiously enough, the Texas Highway Department requires that for their control, apart from pretensioning, we are required to furnish them three cylinders which they ship to the Department in Austin. These cylinders are not steam cured. They are simply cured outside for the first 24 hours and then are stored in water until they are shipped, perhaps at the age of one week. Then they are stored in water at Austin. A good question may be occurring to you: What is the relationship of these cylinders to the conditions under which the members are actually made?

MODERATOR:

I have the same situation in New Jersey and I question it daily. In summary I think the panel agrees, and I know I do, that a laboratory in your plant and under your control, is money well spent. I know that we have actually upped our quality and reduced our costs by spending money on a laboratory in our plant. The next question is one that is also kicked around quite a bit—admixtures. We hear all sorts of claims for admixtures and how they're going to cure all the many problems that confront us daily in producing quality contcrete. How about plasticisers, Mr. Colthurst, do you use them in Canada?

MR. COLTHURST:

We have never used any admixtures in the prestressing field. I don't know exactly why but we sort of go back to the old adage that the best admixture still is cement and that just about covers it.

MODERATOR:

My Lyman?

MR. LYMAN:

We use admixtures in Texas for two reasons, first for workability and second, simply to make sure that you get the concrete out of the chute of the transit mixer faster.

MODERATOR:

Mr. Sutter, what admixtures do you use in Buffalo?

MR. SUTTER:

We don't use any admixtures. That is no criticism of admixtures. We've gotten good results without them. Recently the representative of a company selling admixtures has been spending a lot of his good time in our plant, I feel that if we can produce concrete better and cheaper using an admixture we certainly will do it.

MODERATOR:

Mr. Valore, what experience have you had with admixtures?

MR. VALORE:

We use admixtures and most of the claims that you hear from most of the manufacturers of admixtures are untrue. As long as I'm putting myself in Dutch with the admixture people, I might as well do so with the Portland Cement Association too. I would disagree with my friend from Canada that the only good admixture is more portland cement. That sounds like some propaganda of the Portland Cement Association of about fifteen years ago. Dr. Bathes, I am sorry wherever you are. There—It was Dr. Bates' uncle, I believe, who was my boss at one time at the Bureau of Standards who said that admixtures have a very definite place in producing certain desired modifications and properties in concrete. There are, as we know, more and more producers of admixtures coming to the fore and, of course, many of them state that admixtures are a cure for what ails concrete, and I don't want to generalize.

I would like to refer, if I may, to a publication of Committee 212 of the American Concrete Institute and recommend that all of you get a reprint of the committee report, "Admixtures for Concrete." If any of you want to make a note, its title 51-5. It classifies admixtures, discusses their functions, whether or not these functions are achieved, and I think it would be worthwhile before using any admixture that all of us learn a little something about them. A very important thing to know about admixtures is that you cannot just automatically use an admixture in your plant for the first time without perhaps extensive trial batches with the materials available in your particular area. Some admixtures that will work well with, let's say, 90 per cent of the portland cements in the United States will give completely miserable results in the other 10 per cent. And I might add in that connection that the handling of admixtures is extremely important. If you push that button twice by mistake, you end up with a couple of beams that you have to haul out to the dump. I would like to say then, just one thing about admixtures. We use them largely as retarders. We don't actually save any cement, because maximum and minimum quantities of cement are limited by the State Highway Department on our highway bridges. And finally a word about workability admixtures. Most of them just are not, strictly speaking, workability admixtures when concretes are designed for the same slump. You will get workability only by proper gradation of aggregates or by introducing some additional prime mineral filler or some such material. Again I would like to emphasize, be sure of what you are working with when you go to admixtures. They can help you a lot and they can give you a lot of headaches too.

MODERATOR:

There is one aspect of quality concrete that I think correctly goes under admixtures. That is the use of air-entrainment in a high strength prestressed concrete mixture. This business of using air-entrainment in prestressed concrete, I feel, is more than just taking a basic non-air prestressed concrete mix and adding air to it. There are factors involved that must be taken into consideration by the concrete mix designers. I'd like to discuss some work we did in our laboratory that I feel emphasizes these points. These mixes are typical of mixes we produced in our plant under close quality control. Let's consider a typical strength-age band curve for an air-cured 3 per cent air-entrained, 5,000 pound prestressed concrete mix. If you are trying to release at 4,000 pounds in 24 hours, you are going to have great difficulty in doing it with having air added. And I might say that this condition, the release strength, is harder to obtain as the air increases. At least that is my opinion and what we have found. Also at 28 days, you are very likely going to have difficulty in meeting your 28 day, 5,000 pound strength. For those who might be interested in a little more detail on this particular investigation, I have some reports that will give you more detail. Let's take this same mix steam-cured as the actual beams were steam-cured. Steam-curing helps some, but not enough. You still run into problems in obtaining your release strength and if you have your plant set up on a 24 hour cycle, you are going to have trouble. Here, again, you do hit the 5,000 pound strength pretty well in 28 days, although again, there is not too much of a "comfort factor" involved. Maximum here, I would say, was about 5300 and you should do better than that. The next study was with this basic mix without air. Here, at 24 hours, we're into about 4300 to 4400 psi. At 28 days we're well over 5,000 psi. And I think that if there are any mix designers or State

Highway people in the audience at all, I think you should look into this matter of what you are doing to your basic mix when you add air. And two things that I feel can be done to help the producer to manufacture a concrete you want at the best possible price are (1) perhaps raise your maximum limit of cement factor slightly. For instance, in Jersey our maximum is 7½ sacks. I don't see any real concern about going to 8 and yet it's going to help a lot when you try to hit your strengths using air, especially at the 28 day stage. Now, also, another point, insofar as the producer is concerned, is to lower your release strength requirement to 3,500 if (1) you don't over-stress the member or it isn't cambered too much. These things could, I think, be done with a minimum sacrifice to the quality of product but actually would help the producer maintain a high rate of production and a very economical price. One question that I know is close to Mr. Valore's heart, is one that I have searched for the answer to ever since I've been in the concrete business and have not found them yet. Modulus of elasticity, Mr. Valore. How does it influence quality? Can it be determined readily in plant laboratories and with what degree of accuracy? And how do you determine it?

MR. VALORE:

Unfortunately, the term "modulus of elasticity" has meant various thing to various groups of people and until about two months ago at the ASTM meeting in Boston, except for a method in the concrete Handbook for Engineers, there was nowhere to be found a standard for determining so-called static modulus of elasticity for concrete. The Sub-Committee of C-9 charged with such matters has set up a proposed, tentative method that will be published, I believe, during the next year in the ASTM Standards and this, unfortunately, will not be a secant modulus that we are used to. It will be chord modulus based on a Corps of Engineers requirement. This particular modulus will be obtained as an average of the ratio of stress to strain for the second, third and fourth loadings of a cylinder; the first loading being neglected. This will then be defined as the chord modulus, the datum point will actually be a strain of .0003, or 300 micro inches per inch—at a rate which would correspond to a stress in the neighborhood of that oridinarily considered to be safe in concrete. In many types of prestressed work, strength is specified for pretensioning when it would actually be preferable to have some idea of the amount of deformation we would get and to be able to use a value of the modulus of elasticity, particularly in pretensioning where the immediate elastic loss very definitely must be taken into account, an elastic loss due to shortening of the concrete member. We have worked with various type of gauging techniques. The method that we use and which has, believe it or not, turned out to be quite satisfactory and can be used without much trouble consists of little gauges on the side of the cylinder. We merely attach two gauges to a cylinder, using rubber bands. They are complete Wheatsone bridges in themselves. They are self-compensating as far as temperature is concerned, we hook two of them in parallel, connect them up to a Baldwin or other AC bridge indicator and read the answer directly in micro-inches per inch. It is virtually that simple to do a stress-strain; we require one extra operator as it is to determine compressive strength. Another instrument that might be used is the sonoscope, developed by a Canadian power company. I think we will be making greater and greater use of such means of quality control in the future.

EDITOR'S NOTE:

After panel members had covered the scheduled topics, the session was opened to questions from the floor. The floor discussions were animated and the questions asked touching different phases of prestressed concrete production, testing and instruments for testing, and plant machinery and equipment. Some producers were interested in the experience of others with different typs of portland cement, especially high early strength cement, (Type 3). Majority opinion appeared to favor Type 3 cement for prestressing work.

There was also keen interest in the experience of producers with lightweight aggregate concrete. Several producers reported satisfactory experience with lightweight aggregates during a period of several years, including the production of doubel-T's.

Mr. Valore expressed the opinion that with the combination of high early strength cement and steam curing, it may be possible to pretension lightweight aggregate concrete without excessive losses of strength due to creep and drying shrinkage.

Paul Klieger, senior research engineer, Applied Research section, Portland Cement Association, took issue with a panel member over a statement that the effectiveness of entrained air is based purely on theoretical considerations. "A a matter of fact," Mr. Klieger said, "the effectiveness of entrained air was first recognized from practical experience in the concrete field. The theory came after the practical experience. We have an 18 to 20-year field service record that confirms the necessity of having entrained air in concrete that is to be exposed to freezing and thawing and the use of chemical de-icers."

VOLUME TIGHTLY BOUND

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Quality control in prestressed concrete production

by O. J. Masterman, B.Eng., A.M.I.C.E., A.M.I.Struct.E.

In the preceding article America speaks on Quality Control. Of pertinent and added interest is the following article where England speaks through the voice of one of its outstanding authorities. Reprinted from Magazine of Concrete Research, Vol. 10, No. 29, August, 1958, Cement & Concrete Association, London, England. (Ed.)

SUMMARY

This paper outlines a system of quality control applied to prestressed concrete joists in which the control depends upon daily testing of a random selection of the actual joists. The non-destructive testing includes obtaining the modulus of elasticity of the concrete and the load at the first crack for each joist; corresponding cubes are made for crushing at 1 and 28 days. Results are analysed and the means and standard deviations recorded.

Introduction

Systematic control of the quality of concrete is making rather slow headway in this country. On building sites it is generally confined to an indirect control exercised through cube strengths. In other words, there is usually no measure of the quality of the structure, or even of its components, nor yet of the quality of the concrete which goes into them. The measurements upon which the control depends are made usually on test cubes which are (or ought to be) made from some of the same concrete that goes into the structure; this concrete is, however, differently compacted and has a different curing history from that in the cubes. Strictly speaking, therefore, the cube strengths obtained can only indicate how strong the concrete in the structure would be if the compaction and curing conditions were as good as those of the cubes. Under experienced and keen supervision, the cube strengths are in fact quite a good indication of the quality of concrete in the structure, but the results can be misleading if more care is taken in compacting and curing the concrete in the cubes than in the building. Making a special mix for cubes is, of course, quite wrong.

These difficulties all arise because no totally satisfactory non-destructive test has yet been developed for concrete. There is no intention here to belittle the valuable and extensive work done by the Road Research Laboratory on ultrasonic testing—a non-destructive method by which the quality of matured concrete is judged by the velocity attained by high-speed impulses which are passed through it—but this method is not yet cheap enough, or simple enough, or certain enough for general use. Moreover, it again is indirect because the velocity readings have to be calibrated against cubes.

A good control system should be based upon testing the product rather than cubes and, whilst this is difficult for buildings, it is quite feasible in factories where standard products are made. The author has had the opportunity of setting up and supervising a quality control procedure at the Pierhead factories which make prestressed floor joists. This paper outlines the procedure adopted and discusses the results obtained after several years' use. The joists on which routine tests are made are all of one section, 7 in. deep and 6 in. wide, but waisted to form an X shape, and they vary in length up to about 28 ft. Different spans and loads are accommodated by varying the number of wires in the joists and hence the amount of the prestress. It may be noted that all joists are made with aluminous cement so that very high early strength is obtained and release takes place within 24 hours.

The objects of the control routine are:

(1) to maintain uniformity of quality;
(2) to provide an acceptance test for the joists;
(3) to enable an economical mix to be used with confidence;
(4) to record the results for reference and analysis.

It may be mentioned that the system was first set up in 1952 when there was no draft Prestressed Code and when there was little experience of prestressed concrete production in this country. It is interesting to note that, after six years' working, the original acceptance test requirements are still used. These are:

(1) a modulus of elasticity in the concrete of not less than 5×10^6 lb/in^2 at 1 day;
(2) a load at first cracking of not less than 1¼ times the working load at 1 day;
(3) wire positions and concrete section falling within specified tolerances;

VOLUME TIGHTLY BOUND

(4) ultimate strength not less than 2¼ times the working load under quarter-point loading.

These limits are not absolute rejection limits but are used as an indication that something is probably wrong, and joists which fail to pass or only just pass requirements 1 and 2 are put aside for further examination. The failure test is not done regularly because of the expense, but batches of joists are checked at intervals to ensure that the factor of safety is adequate.

There are two Pierhead factories at which this control system is applied, and another owned by an associated company has adopted similar methods. In the Pierhead factories, from which I have the records, the concrete modulus has seldom fallen below 5×10^6 and the load at first cracking very rarely below 1¼ times working load. If either of these things happens it is a sure sign that something has slipped and that is the warning that is required. The running daily chart (see Figure 1) shows whether a low result is part of a trend or an isolated case, and the chart is drawn up in such a way as to help to see what factors may have caused the trouble. This is dealt with in more detail later.

Whilst the acceptance tests have been useful in providing limits for action, the intention behind them was:

(1) by testing the concrete modulus, to check the quality of the concrete in the joist;
(2) by measuring the load at first cracking, to check the tension in the steel;
(3) by the check on sections and positions of wires, to control camber and lateral straightness;
(4) by testing for ultimate strength, to check the factor of safety.

In a prestressed joist there are only two materials, concrete and steel, and the first object of the control was to make sure that the concrete was of the required quality and that the steel had been given the right tension; the quality of the steel is checked and reported by the makers. We have designed and built a special machine which enables the concrete modulus to be determined on any joist within a matter of minutes as described later. The load at first cracking is regarded as an indicator of the tension in the wires. It cannot distinguish the tension in individual wires but, in the Pierhead system (at the date of writing), all wires are tensioned simultaneously so that the tensions should be equal, provided there are no slips. Wires are fitted with a special clip so that the extensions can be checked and slip at once detected. In the early days when the small bottle anchorages were being developed, there was a certain amount of trouble and the wedges had to be changed frequently. Slips at anchorages are now very rare. Bond failures between wire and concrete are now virtually non-existent. All wire used in these tests was indented 0·2 in. diameter, 100–110 tons/in².

Experience has, however, shown the wisdom of weathering this wire before use, in spite of the indentations.

The mix

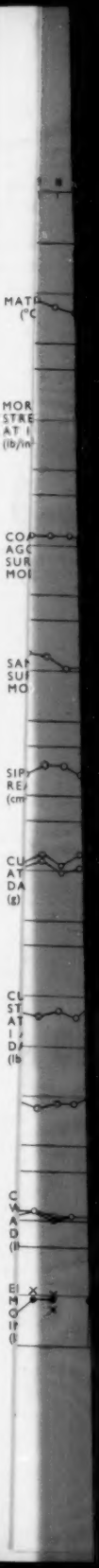
Pierhead undertake to provide in their joists a minimum concrete strength of 7,500 lb/in² at 1 day (release age) and 9,000 lb/in² at 28 days. The control given by the system here described has been so good that these figures can be obtained with a 9:1 mix in summer. Control charts for the first six months of 1957 are reproduced here when, at the works in question, a mix of 7½:1 was being used. It is seen from the chart that the 28 day strength *never* fell below 9,400 nor the 1 day strength below 8,400 lb/in².

The system of control

The joists are pre-tensioned and made on 120 ft long twin lines in steel moulds. A very dry gap-graded mix is used with a water/cement ratio in the neighbourhood of 0·35. Intense vibration is, of course, necessary and external vibrators are fitted to the moulds at 10 ft intervals. The intention is that one joist is tested from each line each day. The test at first cracking therefore gives a check on the tensioning of all the joists in that line. Each day one joist from those selected for testing is picked at random and is called the control joist, and the test figures on the daily chart refer to the concrete in this joist. Only the tests on joists made from Monday to Thursday are included in the records published here because these all have the same testing age, namely 1 day. The control joist is selected by the testing assistant who walks into the shop at any time when casting is going on and takes a sample of the concrete which is being placed in one of the moulds. Cube strengths at 1 and 28 days are taken, the control joist made from the same batch is tested for modulus and first cracking, the maximum and minimum shop temperatures are recorded, the water content of the control batch is measured and a sieve analysis is made of the sand and gravel used. In practice, every line does not get tested every day but the test joists are taken from any lines at random.

Figure 1 shows how these results are set up on the daily control chart which, reading from the top, shows curves for the maturity, the surface modulus of the coarse and fine aggregates, the water content measured by siphon-can, the cube weights, the 1 and 28 day cube strengths, the difference between each pair of cubes tested at 28 days and 1 day and finally the concrete modulus of the control joist obtained from the loading test. An occasional check is also made on the cement to make sure that this is not varying unduly and these results are spotted near the top of the graph. It will be noticed that the cement strength lies pretty close to the concrete cube strength.

All the results are plotted in such a way that a rise in any curve indicates the likelihood of higher strength, and a drop lower strength, in the final cube and modulus figures. When an unusual result appears, the eye can be run vertically up the graph to see which factors are likely to have contributed to the unusual result. Both for cement testing and concrete testing, 4 in. cubes are used, and these are filled in the same way on vibrating tables specially made for the purpose so that the filling is standardized and practically untouched by hand. A low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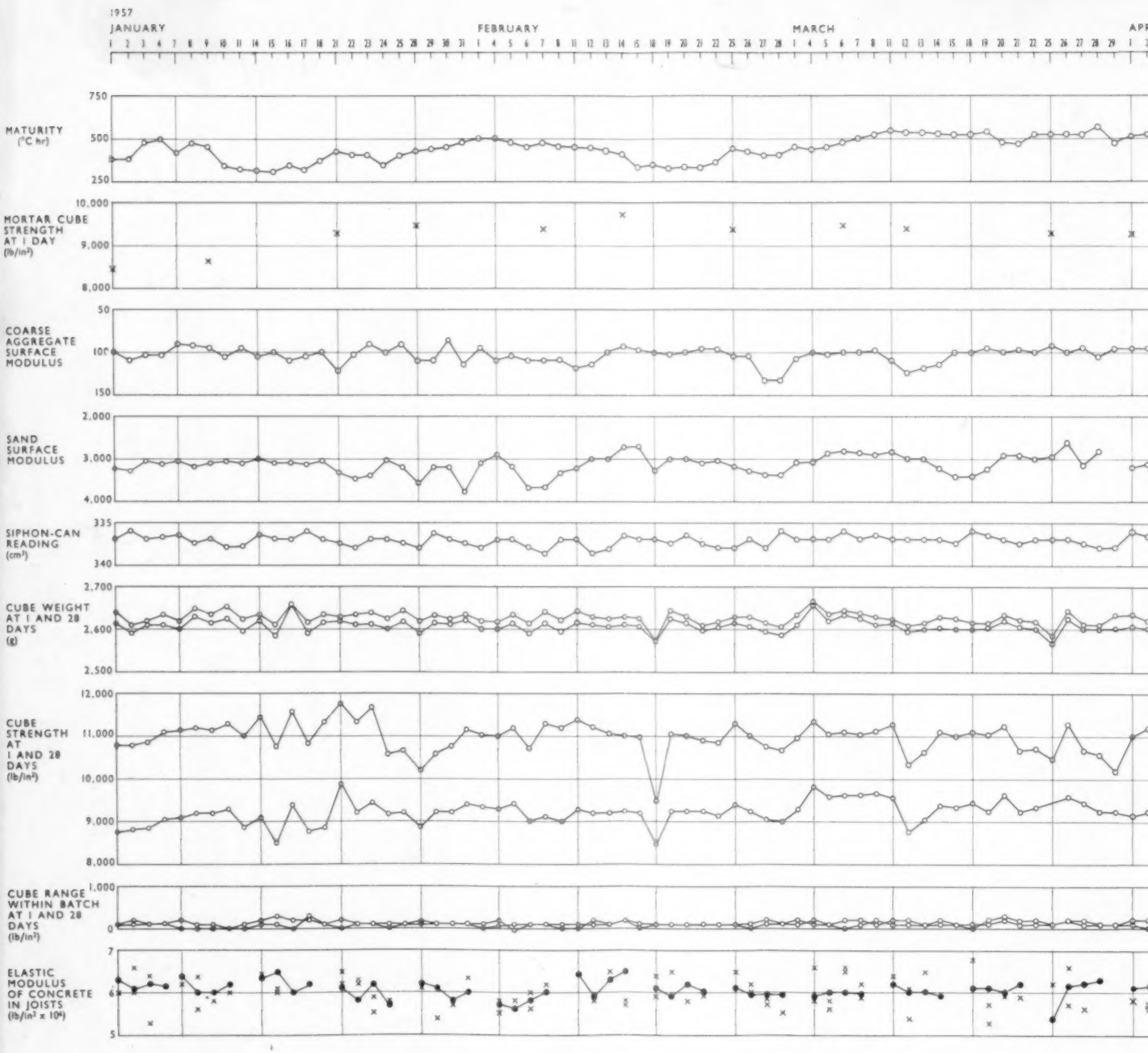
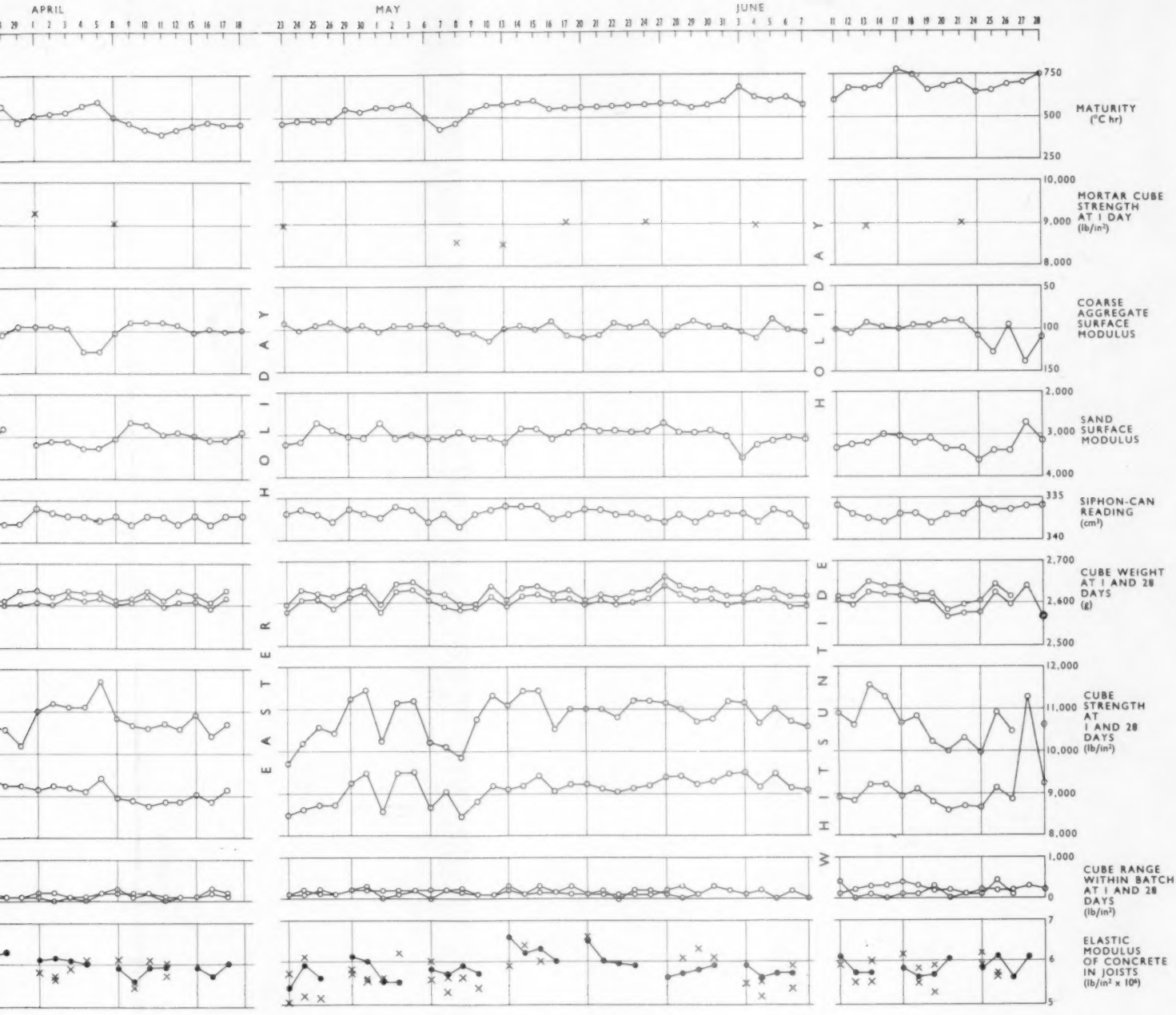


Figure 1: Running daily control chart.



M C R 29 : Masterman. Figure 1

250
240
230
220
0·24

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be weight is often, as would be expected, an indication low strength, and vice versa, although there does not pear to be a useful correlation between cube weight and be strength for the smaller differences. The water con-nt is, of course, one of the most important factors fecting strength and for measuring this the siphon-can s been selected after much study and trial of different ethods. The main advantage of the siphon-can is that e measurement is made on the concrete itself at the oment of placing. The addition of water to the mix is left entirely to the judgement of the mixer man, but the daily check by the testing assistant and his constant supervision work out well in practice, and this seemingly arbitrary method of apportioning water has been found more reliable than measuring it in by weight after checking the water content in the aggregates. It will be seen that the siphon-can readings vary by only 2·5 cm³ throughout the six-month period and this range represents a variation of only 0·02 in the water/cement ratio. Figure 2 shows the required conversion graph.

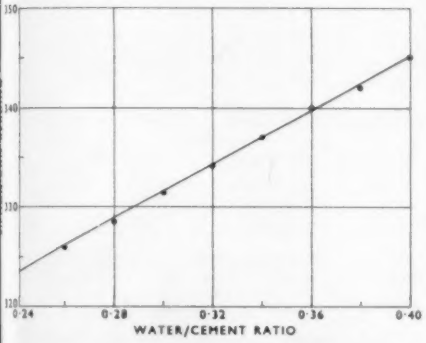
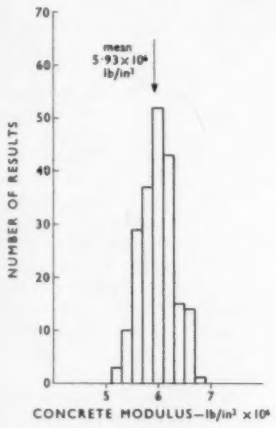


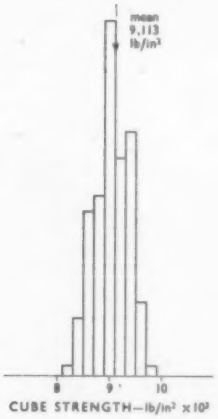
Figure 2: Conversion graph (siphon-can reading to water/ment ratio) used at factory A.

CONCRETE MODULUS OF CONTROL JOISTS, AND CUBE STRENGTHS

Figure 3 shows on the right a histogram of the 1 day cube strengths at factory A from January to June, and on the left a histogram of the concrete modulus of the test joists from the same factory for the same period. The 4 in. cubes are made in batches of four on a vibrating table of Pierhead's own design so that they all get similar treatment in consolidation and the personal element is practically eliminated. They are tested in pairs, two at one day and two at one month, and the histogram gives the mean of 248 separate cubes constituting 124 pairs. The average strength is 9,113 lb/in². The standard deviation over all the 248 results is 307 lb/in², corresponding to a coefficient of variation of 3·37%. The cubes are tested in a machine with a dial and ring for measuring the load, so results are rather lower than would be obtained on a more usual type of machine. The test loading is automatic and corresponds to British Standard require-

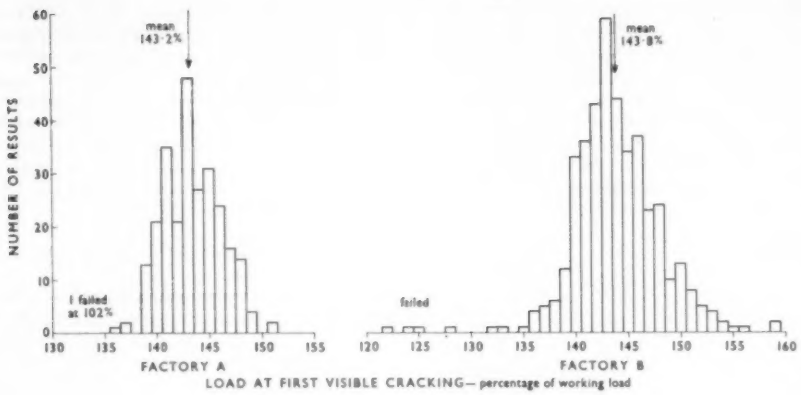


Number of results	204
Mean	5·93 × 10⁶ *lb/in²*
Standard deviation	0·323 × 10⁶ *lb/in²*
Coefficient of variation	5·44%



Number of results	248 (124 *pairs*)
Mean	9,113 *lb/in²*
Standard deviation (over-all)	307 *lb/in²*
Coefficient of variation	3·37%

Figure 3: Histograms of test results at 1 day at Factory A from January to June 1957. Concrete modulus of elasticity of joists and cube strengths.



	Factory A	*Factory B*
Number of results	260	413
Standard deviation	3·91%	3·96%
Coefficient of variation	2·73%	2·75%

Figure 4: Histogram of loads at first visible cracking (as percentages of working loads) in tests on type 8 joists at factories.

ments. Cubes for testing at 1 day are taken straight from the moulds and those for testing at 28 days are kept in water at about 55°F.

The histogram of the concrete modulus measured on the test joists is typical and shows a mean value from 204 joists at factory A of $5{\cdot}93 \times 10^6$ lb/in^2. The standard deviation is $0{\cdot}323 \times 10^6$ lb/in^2 corresponding to a coefficient of variation of 5·44%. The concrete modulus is obtained by taking two readings of deflexion on the straight portion of the load-deflexion curve and converting these by a suitable Table direct to the appropriate modulus figure. The deflexions are read by a dial gauge and 6 ft steel bridge placed in the centre half of the joist under test, i.e. in the portion of the joist that is under constant bending moment. The dial reads to 1/10,000 in. Care is taken to keep the timing constant during loading and reading of the deflexions, and the modulus obtained is practically an instantaneous modulus.

Tests of load at first cracking

Figure 4 illustrates histograms of the figures at first visible cracking as percentages of working loads for all the type 8 joists tested at factory A and factory B respectively It will be seen that there were respectively 260 and 413 results, and that the test eliminated about 1% of those tested. It has in fact been found that a low figure for first cracking is an indication of a probably faulty joist and that this often ties up with a low modulus. Rejections due to slip of steel have not occurred. In those few that have been thrown out as a result of the check on first cracking or the modulus test, the weakness has each time been unsatisfactory compaction at some part of the joist. A noteworthy feature of the results illustrated in Figure 4 is the very close agreement between the two factories, although the aggregates are different, the mixes are slightly different and there are, of course, different testing assistants and supervision.

A check on the figures at first cracking

At the time this control system was set up, it was not known how the figures at first cracking would vary or what were reasonable average values for each type of joist. It will be remembered that the main object of this test was to establish that the tension in the wires was correct. The graph in Figure 5, first suggested by R. H. Squire, has been drawn to clear up this point and others mentioned below. The graph plots the change of stress (M/Z_b) in the bottom flange between release and first crack against the different types of joist, and it has been set up from the argument and equation shown in the Appendix. In drawing the graph, all results at first cracking from both factories for the whole year 1957 have been included, and the number of tests averaged for each point on the graph is shown in brackets. The ordinates are the actual moments at first visible cracking divided by Z_b (the section modulus) and, as this figure is practically constant for all types, the graph virtually shows the test results at first cracking plotted against joist types.

Two lines have been drawn. These are approximate best lines and have not been derived mathematically. The upper one represents the results at factory A from January to June and the lower one the results from both factories for the whole year. Plots for types 2 and 7 are omitted because very few have been made. Type 1 joists are not manufactured.

Other particular aims behind this graph were to see

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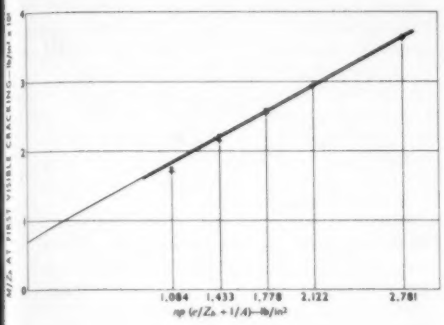

Figure 5: Relation between M/Z_b and $np(e/Z_b+1/A)$. M_b = actual moment at first visible cracking under test. Z_b = section modulus. Points plotted are averages of a number of tests.

- *tests at factory A, January–June 1957.*
- \+ *tests at factories A and B, January–December 1957.*

whether the points came out on a straight line, since the equation shown in the Appendix is linear, and to see what intercept was made on the ordinate through the origin as this would represent the tensile strength of the concrete in the bottom flange. It will be seen that the test figures of types 4, 5, 6 and 8 fall reasonably well on a straight line and these are the types for which there are most results. Types 3 and 2 are very much under-reinforced and Squire had already pointed out* that in such cases the apparent concrete tension would be low. Type 3 figures have therefore been ignored in drawing the best lines on this graph, and the resulting line for factory A shows a concrete tension figure of 700 lb/in². This compares reasonably well with rupture modulus tests made at the factory on 16×4×4 in. unreinforced beams which have varied between 700 and 800 lb/in². This graph would, of course, be affected by the prestress losses, but all the figures are taken from tests on joists 1 day old, so it might be claimed that the effect of losses on the linearity of this curve would be slight.

It is often argued that figures taken at first cracking are rather unreliable because different observers will not see the cracks at the same time. In fact, under the procedure for this test, the operator watches a very sensitive deflexion dial as the load is increased by equal increments. He thus gets quite clear warning when he is approaching the point at which the first crack can be seen. The uniformity of the results from two different factories given in Figure 4 serves to show that this argument may be ignored. Indeed, the remarkable similarity of results from two different observers at two different works is one of the unexpected features of this control system. Great pains were taken at one period to establish the moment at first cracking graphically from load-deflexion curves. Rather surprisingly, it has been found that this method does not locate the first crack more precisely than the visual method which has now been regularly adopted.

*SQUIRE, R. H. Some important aspects of the elastic modulus of concrete. *The Structural Engineer*. Vol. 21, No. 6. June 1943. p. 233.

The section check

The section check is made at the time when each joist is tested by applying a drilled Perspex plate to the ends of the joist. There is a template for each different type. The drilled holes fit over the wires and the permitted tolerances of the concrete section are scratched on the template. This test checks whether the wires are correctly positioned relative to one another and to the concrete section, and whether the section is within the allowed tolerances.

The modulus test

Figure 6 shows a view of one of the joist-testing machines specially developed for getting very quick values of the concrete modulus and for testing load at first cracking and ultimate strengths. The joist is lowered on to the machine upside down and the load is applied at the quarter-points by the central ram through a proving ring and a small steel spreader beam. The proving ring and dial are checked annually and have been found to keep in good shape. On top of the joist under test is laid a 6 ft steel bridge with another dial gauge which records deflexions (to 1×10^{-4} in.) in the centre part of the joist, which is under constant moment. It is found that with these joists, as with other prestressed products, the load-deflexion curve is virtually straight up to and beyond working load. The method of obtaining the concrete modulus is, therefore, to take a reading of the deflexion at one-quarter and at full working load. This gives the slope of the curve from which, with the aid of a suitable Table, the modulus can be read off at once.

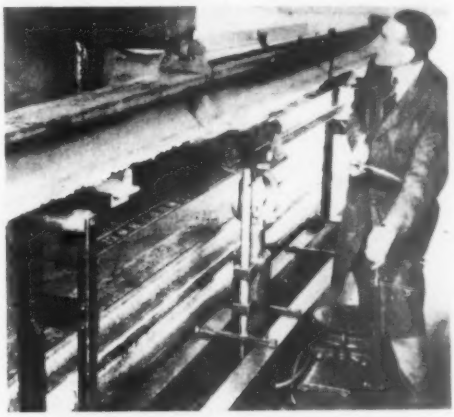

Figure 6: Joist-testing machine.

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Comment

In looking back over the test results of which typical extracts are shown in this paper, perhaps the two things which have been most unexpected are:

(1) that the results of cube tests, joist modulus and first cracking tests from two different factories over a long period have lined up so closely;

(2) that uniformity in all the above items has been so good.

The two factories, as mentioned, use different aggregates, and each has, of course, a different testing assistant. All the test figures are obtained from the regular daily reports and the testing is done by operators who, although trained for the work, have had no previous research or concrete background. All tests made under this control system during the year 1957 are included in the diagrams and analyses presented here. It may be noted, however, that even among the small number of failures which the system threw out (see Figure 4) no joist in fact failed at a load below the working load.

Reference to Figure 1, which shows the daily record at factory A, indicates that, although the sand received has varied considerably in grading, ranging at times over all the zones, it does not appear to have had a recognizable effect on the uniformity of either cube or joist modulus results. An exception occurs on 27th June when the grading of both aggregates was suddenly changed and where the effect was to obtain an exceptionally high 1 day cube strength of 11,300 lb/in^2. It has, however, been the practice when the sand grading varies much to mix in a proportion of special sand so as to keep the grading within reasonable limits. The degree-hours of curing on the top line are obtained by adding 10°C to the average of the maximum and minimum temperatures for the day and multiplying by 24. The cube weights, especially where they are excessively high or low, are as a rule clearly reflected in the cube strengths. The 28 day cube strengths follow reasonably well above the 1 day strengths. On the bottom line, control joist figures are indicated by dots and those from other joists by crosses. Confidence limits were for some time worked out and marked on these graphs, but the results have kept so well within them that this is no longer done.

Conclusions

(1) The conclusion reached by Pierhead, after operating this control system for over five years, is that it has been very well worth while, because it has enabled an economic mix to be used with confidence, and because in practice it quickly reveals any deterioration in quality in manufacture and generally gives an indication of the cause.

(2) The control system imposes a constant watch on the tensioning, mixing, filling and curing of the joists and this all has its effect in helping to maintain uniformly high quality.

(3) The facts that no rejections or failures have been traced to faulty tensioning and that steady high concrete strengths can be relied upon give confidence in manu facturing the larger prestressed units where a system o testing the actual products in the way here describe would not be feasible because of the expense.

(4) The high uniformity of the figures at first crackin has enabled standards to be set for each type of joist an has given confidence in the uniformity of the pr tensioning.

ACKNOWLEDGEMENTS

Acknowledgements are due to Pierhead Ltd for pe mitting these results to be published and to Kenneth Do B.Sc., A.M.I.C.E., and J. L. Hutter, A.M.I.Struct.E., wh helped the author greatly in setting up and running th control system; also to R. H. Squire, M.I.Struct.E., fo much valuable advice and to R. Dawn and J. Edwards fo their care and reliability in doing and recording the dai tests.

APPENDIX

Graphical analysis of loads at first cracking for Pierhea X 7 in. prestressed concrete joists.

Notation

A = equivalent area of concrete and steel section
p = mean force in each wire at age of test
n = number of wires in joist
e = eccentricity of centre of gravity of steel from equivalent section centroid
F_t = modulus of rupture of concrete
F_b = induced prestress at bottom face at age of test
M = experimental moment at first visible crack
Z_b = modulus of equivalent section at bottom face

Theory

We have $M = Z_b(F_b + F_t)$

and $F_b = npe/Z_b + np/A$

Therefore $M = Z_bF_t + npZ_b(e/Z_b + 1/A)$

or $M/Z_b = F_t + np(e/Z_b + 1/A)$

We can plot a graph of M/Z_b against $np(e/Z_b + 1/A)$ it will make an intercept F_t on the M/Z_b axis.

Type	n	Z_b (in^3)	e (in.)	A (in^2)	$np(e/Z_b+1/A)$ (lb/in^2)
2	2	44·23	1·610	28·03	729
3	3	44·74	1·603	28·16	1,084
4	4	45·18	1·596	28·28	1,433
5	5	45·33	1·589	28·41	1,778
6	6	45·52	1·582	28·53	2,122
7	7	46·05	1·575	28·66	2,446
8	8	46·25	1·568	28·79	2,781

THIRD CONGRESS OF THE

FEDERATION INTERNATIONALE DE LA PRECONTRAINTE

BERLIN 1958

List of Papers

ABBREVIATIONS TO INDICATE THE COUNTRY OF ORIGIN

(A) Austria
(AUS) Australia
(B) Belgium
(CH) Switzerland
(CS) Czechoslovakia
(D) Germany
(DK) Denmark
(E) Spain
(F) France
(GB) Great Britain
(IND) India
(I) Italy
(NL) Netherlands
(NZ) New Zealand
(PL) Poland
(R) Roumania
(SU) U.S.S.R.
(SF) Finland
(USA) U.S.A.
(ZA) South Africa

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3	J. J. Rieve (D)	Zur Tragfähigkeit und Formänderung von Spannbetonquerschnitten
4	A. Páez (E)	Tests to destruction, by bending, of prestressed concrete beams

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3	J. Ipsen (DK)	Danish experiments on freezing of grout
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12	Ryszard Kowalczyk und Jerzy Zieliński (PL)	Spannkraftverluste und Fehler bei ihrer Bewertung
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17	L. Schmerber (D)	Erfahrungen mit Spannbeton-Prüfgeräten nach dem SD-Verfahren
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SESSION III

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2	J. W. A. Ager and E. W. H. Gifford (GB)	Progress in the manufacture and use of precast prestressed concrete in Great Britain
3	Harold A. Price (USA)	The current status of the art of prestressing as practised by American manufacturers
4	Ben C. Gerwick, Jr (USA)	Plant manufacture of large pre-tensioned bridge girders
5	Eugene M. Smith (USA)	Precast prestressed construction in the U.S.A.
6	Jar. Vondráček (CS)	Fabrication de tuyaux en béton précontraint
7	B. Staněk (CS)	Ateliers ambulants de préfabrication pour les ponts en béton précontraint
8	Andreas Kirchknopf (D)	Weitgespannte Spannbeton-Fertigteile für den Industriebau
9	G. D. Base (GB)	An investigation of transmission length in pre-tensioned concrete
10	E. H. Ratz, M. M. Holmjanski and V. M. Kolner (SU)	The transmission of prestress to concrete by bond
11	W. M. Sutherland (NZ)	Prestressed concrete bridges in New Zealand
12	P. W. Abeles (GB)	A new bridge deck (the 'wafer' slab) used by British Railways, Eastern Region
13	A Special STUVO Committee (NL)	Precast prestressed concrete in the Netherlands
14	M. R. Roš (CH) mit G. E. Lax und P. Donner (SF)	Hohe Rahmenkonstruktionen in Elementbauweise mit vorgespanntem Beton
15	Ricardo Barredo (E)	Construcción de vigas ligeras para cubiertas de grandes luces
16	Mircea Halmagiu, Laur Scripcă und Gert Kessler (R)	Spannbeton-Fachwerkträger aus vorgefertigten Bauteilen
17	Z. Zieliński (PL)	Entwurfs- und Verwendungsprobleme wirtschaftlicher Spannbetonträger
18	Rudolf Bührer (D)	Herstellung und Zusammenbau von Spannbetonschalendächern aus Fertigteilen
19	P. Mlosch (D)	Steildächer in Spannbeton

THIRD CONGRESS OF THE
FEDERATION INTERNATIONALE DE LA PRECONTRAINTE
BERLIN 1958

SESSION I GENERAL REPORT

Developments in design methods

by H. Rüsch

THERE HAVE BEEN many contributions submitted on this subject, most of them dealing with the dimensioning of prestressed concrete structures under the action of compression, bending, shear, and torsion. Some deal with special structural forms and the consideration of prestress in the building codes.

THE COMPRESSIVE STRENGTH

In several contributions the influence of biaxial stress on concrete strength is discussed. The problem concerned is of great importance, but unfortunately it is still not yet fully explained. Since the valuable investigation of Mohr, A. Föppl, and Karman, published a long time ago, only limited progress has been achieved.

Glomb of Poland (No. 1), with the aid of a machine specially built for the purpose, checked the influence of biaxial compressive stress on the strength testing of a great number of 20 cm cubes. He found an increase in strength of 25–50% in comparison with the normal cube strength. Even if one compressive stress was only half the other, the increase in strength still reached 11–25%. All cubes were destroyed through cracking in the plane of the compressive stresses. By the insertion of reinforcing bars crossing these cracks, the strength was increased another 24–50%.

Glomb points out that, according to these results for biaxial compression, the admissible stress could be increased. He especially refers to two-way concrete slabs, prestressed concrete highways, and shells.

The results which Glomb reports were obtained from concrete cubes. The friction developing on the loaded surface inhibits the transverse elongation and produces an embracing effect. Therefore, even in the normal concrete cube test, there occurs a triaxial state of compressive stress, which is why tests using cubes always give greater strengths than tests using cylinders. In the biaxial test this effect is increased. Here the friction on the end surfaces produces the same effect as the transverse reinforcement used by Glomb. It would be advisable to continue these tests using disk-shaped specimens; in these, as in cylinders, the embracing effect becomes negligible in the zone of failure. The strength values

will then be lower, but presumably a better ratio of single and biaxial loading will be achieved.

The biaxial strain is also important in beams, above all for the shearing strength, but in a beam there is a compressive stress in one direction and a tensile stress in the other. In the contributions by Páez (No. 8) and Walther (No. 9) simplified formulae for the strength for this range are derived from Mohr's hypothesis of failure. There is no doubt that in this very range further investigations are necessary to define the concrete strength under such conditions.

Kluz of Poland (No. 24) reports experiments on columns prestressed with wires 1·5 mm in diameter. The ultimate strengths reached were from 43 to 80% higher than the computed values. He explains this fact by a triaxial state of stress which causes compressive stresses of 20,000 to 40,000 kg/cm^2 in the prestressed reinforcement. Unfortunately the calculation of the deformations carried out to prove these surprising values is incomplete, and we would rather like to assume that the increased ultimate strength is due to an underestimate of the concrete strength.

THE FLEXURAL STRENGTH

The theory of flexural strength of prestressed beams presented by Ramaswamy and Narayana of India (No. 2) is based on simplified assumptions similar to those suggested on former occasions for the design of reinforced concrete beams. The authors consider only the condition of equilibrium, neglecting the compatibility of deformations. They justify this approximation by the lack of knowledge about the value of the ultimate concrete strain and of the steel strain at the position of cracks.

The paper by Rieve of Germany (No. 3) contains some important considerations on the influence of the ratio of reinforcement, the quality of steel, and the cross-sectional shape, on the critical load for cracking and failure of beams, based on an already published theory[3] and a diagram especially derived from it.

The first paper by Páez of Spain (No. 4) describes eleven interesting tests on prestressed concrete beams of 13·4 m span and 21 × 40 cm cross-section which have been carried out to study the influence of the magnitude of prestress, the ratio of reinforcement, the non-prestressed reinforcement in the compressive and tensile zones of the beam, and the bond. The loading consisted of 2–6 concentrated loads. With only one exception a pure flexural failure was obtained. The author compares the obtained ultimate moments with his theory presented at the First Congress in London. Since this theory only satisfies the equilibrium conditions he has to make assumptions about the steel stress at failure in order to get satisfactory agreement.

The paper by Wittfoht of Germany (No. 5) is chiefly interesting because of the careful evaluation of experiments comparing the influence of static and dynamic loading. The beams were unusually short (2·0 m span and 25 × 30 cm cross-section). On the basis of the measurement of deformations he tries to trace the weakening of bond and the corresponding redistribution of stresses with an increasing number of load cycles. Under dynamic loading, he observed a larger distance between cracks and a weakening of bond at an earlier stage of loading. In the static test the compressive zone of concrete failed; in the fatigue test, the steel. For the given dimensions and cracked concrete section the ultimate load reached in the fatigue test (minimum stress about 0) amounted to 48% of the load in the static test.

Pietrzykowski of Poland (No. 6) reports tests on prestressed beams made from natural stone. He points out that in some countries concrete of high strength is more expensive than natural stone of an average quality which is easy to machine. He describes tests on beams, 5 m in length and 24×40 cm in cross-section, which were composed of five blocks fitted together. The prestressing tendons were laid in cut-in grooves, which were filled with mortar to create bond. In some of the beams the stones were butted against each other, while in others the joints were grouted or filled with roofing paper. Unfortunately, the experimental results are not reported completely enough to enable a comparison with the strength of prestressed concrete to be made.

Mikhailov of the U.S.S.R. (No. 7) examines the advantages of composite structures which consist of precast prestressed concrete members and concrete cast *in situ*. In the power plant at Kuibyshev the roof over the turbine entrances was built in this manner. The two materials are used together in order to make possible a better utilization of the tensile strength of the concrete cast *in situ*. Generally, the branch of the stress-strain diagram of concrete that lies behind the point of highest tension cannot be utilized because above 0·01 % elongation the tensile force taken by the concrete decreases. Through the combination with prestressed concrete, in which the highest tensile stress occurs at about 0·05 % elongation, a composite structure can be obtained in which no cracking occurs when the tensile stress exceeds the strength of the concrete cast *in situ* because with further increasing elongation the total tensile force of the composite system still increases.

Mikhailov explains, on the basis of the stress-strain diagram of the composite material, under what circumstances the occurrence of tensile cracks in concrete cast *in situ* could be delayed up to approximately 0·03 % elongation by the combination of precast and prestressed members. He also describes tests performed on beams 5·5 m in length and 36×80 cm in cross-section which were assembled from four prestressed concrete girders and concrete cast *in situ*. These tests confirmed the favourable effect of the composite structures on the tensile strain at failure of concrete cast *in situ*.

The calculation of the flexural strength of prestressed concrete seems still to be a problem. Formulae are still being used which only fulfil the equilibrium condition but not the compatibility of the deformation. Such formulae cannot do justice to the problem of prestressing, based itself on a pre-deformation of the steel. Such formulae can only yield satisfactory results in the range of under-reinforced beams where it is sufficient to estimate the lever arm of the internal forces approximately. Various authors [1, 2, 3] have shown that the compatibility of the deformation can be accounted for in the theory of bending in the simplest manner. However, different points of view existed until recently about the deformation law to be used for concrete. As a result of work now published[4, 5, 6] and through the activity of the Comité Européen du Béton, the problem has now been solved to the extent that a satisfactory agreement with test values has been obtained with uniform formulae in the whole range from under-reinforced to over-reinforced sections. The range of over-reinforced sections occurs much more frequently than is generally assumed. It is usually overlooked that all structures which are loaded simultaneously by a bending moment and a compressive force lie in this range.

If any differences of opinion still exist today about the law of deformation, they are based on the false assumption that the ultimate strain of concrete is constant. It is easy to prove that the magnitude of the strain at failure not only depends on the position of the neutral axis, but also on the shape of the cross-section. At the moment of failure the stress-strain diagram of the concrete is always utilized up to the edge-strain (Figure 1) that gives the maximum moment at failure. The value of the edge-strain that leads to the largest moment differs with the position of the neutral axis. The cross-sectional shape of the compressive zone has an even greater influence on the ultimate strain which leads to the maximum moment. In Figure 1 the results of an evaluation of the same stress-strain diagram for characteristic shapes of cross-section and two limiting values of the position of the neutral axis are represented by indicating the concrete strain at failure and the corresponding distribution in the compressive zone. These results show that for the same concrete there may exist very different distributions of stress and different values of strain at failure.

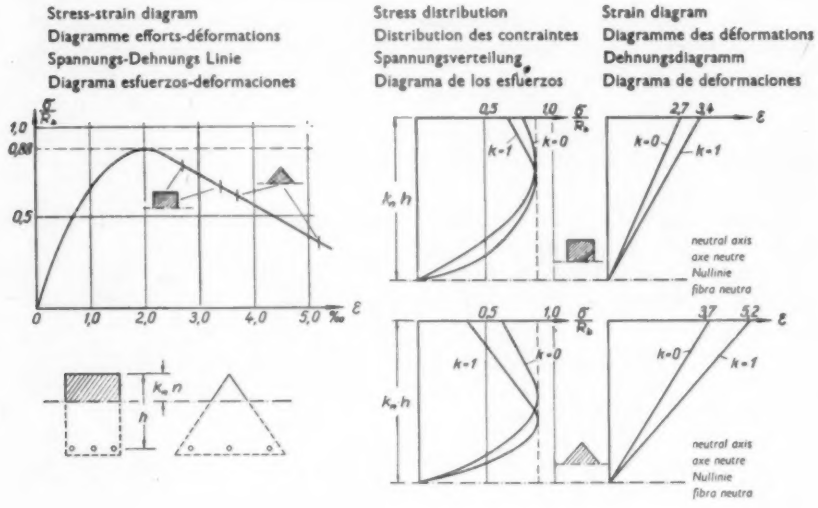

Figure 1

THE SHEAR STRENGTH

In the theory proposed by Páez of Spain in his second paper (No. 8) an attempt is made to calculate the principal stresses caused in the compression zone of bending by the combined action of moment and shear force. He attempts also to estimate the concrete strength corresponding to this state of stress on the basis of Mohr's hypothesis of failure and is then able to specify those combinations of moment and shear force which lead to failure. Páez limits the applicability of this theory, which stems from the determination of the state of stress using very simple assumptions, to that range for which the compressive strength under biaxial strain does not decrease under 0·6 R_b. Beyond this limit he then completes his diagrams empirically, on the basis of experimental values. According to

Páez the ultimate load of beams without shear reinforcement only depends on the tensile strength of concrete.

Walther of the U.S.A. (No. 9) also proceeds from the assumption that the strength of the compression zone of bending mainly determines the shear failure. Contrary to Páez, he takes into consideration only a compression failure of concrete. He determines the compressive strength for the biaxial state of stresses in an approximate way which differs from Mohr's hypothesis of failure. He also takes into account the contribution of compression or transverse reinforcement. For the distribution of bending and shearing stresses he suggests simplified assumptions and he then calculates the position which the neutral axis takes at failure on the basis of further, somewhat arbitrary, assumptions for the deformations.

Warner and Hall of Australia (No. 10) consider the occurrence of the first shear crack to be the real cause of rupture because, with prestressed structures, the failure always follows very quickly after this crack. They calculate, therefore, the principal tensile stress which leads to the shear crack on the assumption that the stress-strain diagram is triangular. This calculation is carried out for the cracked as well as for the uncracked section. They then determine a formula, based on the tests, for the 'modified tensile strength' leading to the shear crack.

Evans and Hosny of Great Britain (No. 11) analyse a large number of new beam tests which were carried out at the University of Leeds. They attempt to explain the results on the basis of the shearing stress leading to failure, and give an empirical formula relating this stress to the shear span, the concrete strength, and the magnitude of prestressing. A similar formula is presented for the shearing moment at failure which depends on the concrete strength and the degree of prestressing. The authors also furnish empirically derived formulae for taking into account the shear reinforcement and for determining the principal tensile stress leading to the shear crack.

Abeles of Great Britain (No. 12) reports new shear tests on a slab composed from precast parts and concrete cast *in situ* and adds an interesting discussion of the problem on the basis of a thorough knowledge of the present position.

In his contribution, Visser of the Netherlands (No. 13) discusses thoroughly the different factors that influence the shear strength and he recommends further experiments.

In the above contributions the causes of rupture are described quite differently. Páez thinks that in beams without web reinforcement the failure is caused by the principal tensile stress occurring in the zone of flexural compression. Walther wants to explain all shear failures by the crushing of the concrete in the zone of flexural compression. Warner and Hall describe two causes of rupture, the shear-compression failure developing slowly in beams of small shear span and low prestress and the sudden failure following the first shear crack in beams that are highly prestressed. Evans distinguishes between shear-compression failure, diagonal crushing of the web, and the possibility of a sudden shearing of the compression zone. Abeles supports the opinion of Laupa, Siess, and Newmark[9], who describe, for decreasing shear spans, a transition from pure flexural failure through diagonal shear failure and shear-compression failure under the joint action of moment and shear force, to shear failure proper with loads acting close to the supports.

However, all authors agree that shear failure is produced by the joint influence of moment and shearing force. Thus the present method of dimensioning for each force acting separately has to be abandoned in any case. The shear failure is closely related to the flexural failure; that is to say, in both cases the rupture is brought about by the overcoming of the tensile strength. With flexural failure this usually happens owing to yielding of the steel, with shear failure through diagonal cracking of the concrete. In both the failure will be completed in the same way – by crushing of the concrete. This compressive failure can occur in different places, as will be shown more clearly in the following.

Most frequent is shear-compression failure. It is caused by a rise of the neutral axis in the range of the concentrated load – this can be observed in all tests – which greatly reduces the area of the compression zone. Evans, Walther, and others[8] think that this reduction is brought about by a discontinuity in the strain diagram. But it can more easily be explained by the fact that the concrete element, limited by two cracks in this range, is longer on the tension side than on the compression side.[7] Consequently the location of the neutral axis can be defined by the formula:

$$k = \frac{\epsilon_b}{\epsilon_b + \frac{b}{a}\epsilon_s}$$

The ratio b/a depends on the pattern of the cracks according to Figure 2.

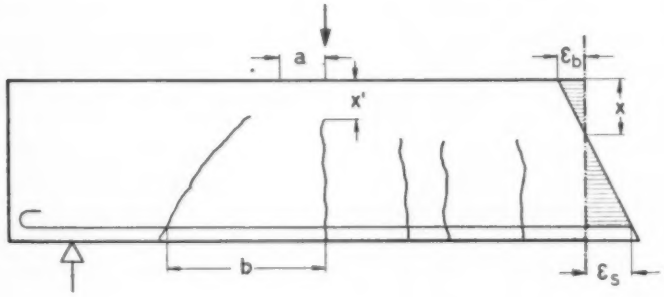

Figure 2

Evans and others[3] showed that this reduced compression area can bear unusually high compressive stresses. But this is only true if a biaxial state of compression is caused by the concentrated load acting at the edge of the compressive zone, thus raising considerably the compressive strength, as Glomb has shown.

Crushing of the web can generally be expected with the thin webs of T- and I-profiles. As soon as bending cracks appear the beam behaves more or less like a frame with a tie, the brace of which causes very high compressive stresses in the web (Figure 3). The lenticular expansion of these compressive stresses brings about transverse tensile stresses, which lead with increasing load to the diagonal shear crack. In the column so split, the compressive force acts eccentrically;

therefore, the maximum compressive stress jumps to about double size. Consequently, if the web is not reinforced by strirrups, this shear crack is followed almost immediately by rupture.

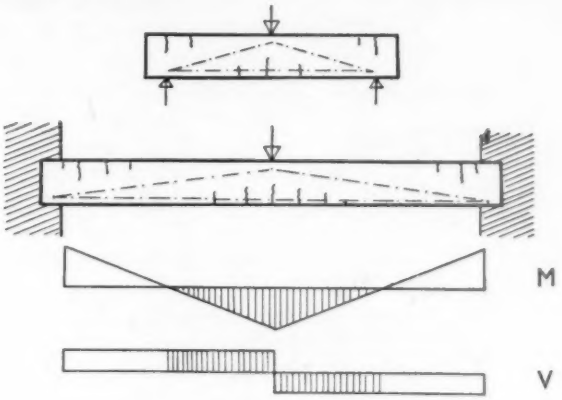

Figure 3

Failure by shear proper occurs when the concentrated load acts very close to the support. Its cause can lie in high compressive stresses as well as in a failure of bond. Failure of bond is especially frequent because, after the development of a diagonal shear crack, the anchorage length available to the reinforcement becomes very short.

Thus, the circumstances that most frequently lead to rupture seem to be fairly certain but we are still far from being able to establish a satisfactory theory to explain all failures. This can only be done if we consider that, in contrast to bending, almost all shear failures occur in a zone full of stress disturbances. The following consequences are difficult to estimate.

(a) At the points of loading or supports high interference stresses develop.

(b) All loads applied from outside cause a biaxial state of compressive stress, with a corresponding increase of strength, thus augmenting the ultimate moment. If the load is applied by means of cross-beams, this favourable effect is missing. Loads hanging on the lower edge of the beam lead to a big decrease of the ultimate moment.

(c) The state of stress is not yet unequivocally defined by the ratio M/Vh. The stress distribution developing after the appearance of the tension cracks depends directly upon the location of the points where the supporting forces act. In Figure 3 a simply supported beam is compared with a restrained beam. Although in the middle range M and V take the same values, the 'frame effect' is much more disadvantageous in the restrained beam, which at the point of zero moment is supported by shear stresses and not by real supporting force at the lower edge. American experiments[10] showed that a considerable drop of strength will be observed in the restrained beam.

(d) Further effects that are difficult to trace are caused by the cracking. Where the first crack appears is largely a matter of chance, and therefore the stress diagram which is affected by the cracks is also subject to haphazard effects.

TABLE 1

Formulae proposed for the calculation of shear strength

Páez

$$m_s = f(q, t)$$

$$q = 0{\cdot}9 p R_p / R_b$$

$$t = \frac{V}{bhR_b}$$

Walther

$$m_s = k_n (1 - \tfrac{1}{2} k_n)$$

$$k_n = \frac{\sigma_p + 1{,}960 \sqrt{h/\kappa' d}}{R_b/p + 3{,}300 \sqrt{h/\kappa' d}}$$

Warner & Hall

$$m_s = \frac{aJ}{h^2 R_b S} \sqrt{(\tfrac{1}{2} p\sigma_p + R_t)^2 - (\tfrac{1}{2} p\sigma_p)^2}$$

$$R_t = \alpha(R_b)\ \beta(a/h)$$

$$\alpha(R_b) = 0{\cdot}043\ R_b \qquad (R_b > 350\ \text{kg/cm}^2)$$

$$\alpha(R_b) = 0{\cdot}096\ R_b - 1{\cdot}50 \times 10^{-4}\ R_b^2 \qquad (R_b < 350\ \text{kg/cm}^2)$$

$$\beta(a/h) = 1{\cdot}0 \qquad (a/h > 1{\cdot}5)$$

$$\beta(a/h) = 6{\cdot}85 - 3{\cdot}90(a/h) \qquad (a/h < 1{\cdot}5)$$

Evans & Hosny

$$m_s = k_1 (0{\cdot}10 + 2{\cdot}1 \times 10^{-3}\ p\ \sigma_p)$$

$$k_1 = 1{\cdot}5 - 0{\cdot}9 \times 10^{-3}\ R_b$$

Zwoyer & Siess

$$m_s = p\sigma_{su}\left(\frac{1}{R_b} - \frac{k_2 p\sigma_{su}}{k_1 R_b^2}\right)$$

$$\sigma_{su} = \frac{K\epsilon_{bu} k_1 R_b/p}{\epsilon_{su} - \epsilon_{sp} - \epsilon_{bp} + K\epsilon_{bu}}$$

$$K\epsilon_{bu} = (0{\cdot}40 + 7{\cdot}15 \times 10^{-5}\ \sigma_p)\ 10^{-3}$$

$$k_1 = 1{\cdot}50 - 10^{-3}\ R_b; \quad k_2 = 0{\cdot}42$$

NOTATION

$m_s = \dfrac{M_s}{bh^2R_b}$

V = shear force

R_b = crushing strength of standard cylinders

R_p = tensile strength of tendons

R_t = modified tensile strength

h = effective depth of beam

a = shear span

S = first moment of compression zone

J = moment of inertia

d = bar diameter

$p = A_s/A_b$ = ratio of steel to concrete

σ_p = tension in steel

k_n = coefficient for height of compression zone

k_1 = coefficient of stress distribution

k_2 = coefficient for stress resultant

$\epsilon_{su}, \epsilon_{bu}$ = steel and concrete strain at failure

$\epsilon_{sp}, \epsilon_{bp}$ = steel and concrete strain due to prestress

κ' = coefficient of roughness (5·5 for plain round bars, 1·5 for rusted strands)

Under complicated conditions it is naturally tempting to look for empirical solutions but it will be difficult to succeed in expressing the multiplicity of phenomena in a simple formula, even if we add a number of constants to it in order to facilitate the adaptation to the experimental results.

Table 1 gives a list of the formulae offered up to now for the determination of shear strength of prestressed concrete. As most of the authors regard the ultimate shear moment as the best scale for the determination of the shear strength, all the formulae have been transformed in such a way that they give the shear moment at failure in the same notation and in the same units (kg,cm). The formulae are valid in the given form only for rectangular prestressed concrete beams without web reinforcement.

As previously mentioned, the coefficients appearing in the formulae have been chosen by the authors in such a way that agreement with the test results is satisfactory. In Figures 4 and 5 the formulae have been analysed for average conditions. As we see, there are considerable differences, perhaps mainly owing to the fact that the various theories are based on different test data. Thus, we are still far removed from a common view.

Evans and Hosny also developed formulae for the shear stress leading to rupture, and for the principal tensile stress producing the shear crack. I wish we could take it for granted that this principal tensile stress at least will assume a constant value, because the stress distribution preceding cracking can definitely be calculated; but experience has shown that the shear crack appears at very different stresses. Obviously, the interferences previously listed also have an effect here.

Before a satisfactory theory of shear rupture can be established, all the different influences have first to be sufficiently clarified by further tests. To these influences belong the shape of the cross-section, the degree of prestressing, the

shear span, the type of load and support, the web reinforcement, the strength of the concrete, and the strength under combined stresses, to mention only the most important factors.

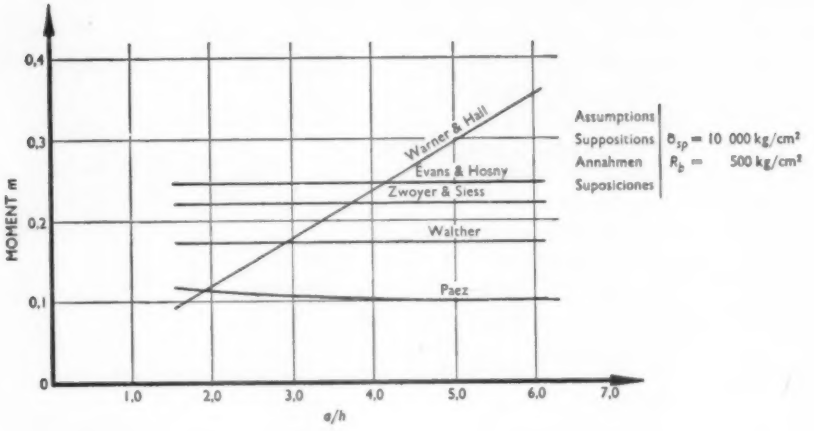

Figure 4

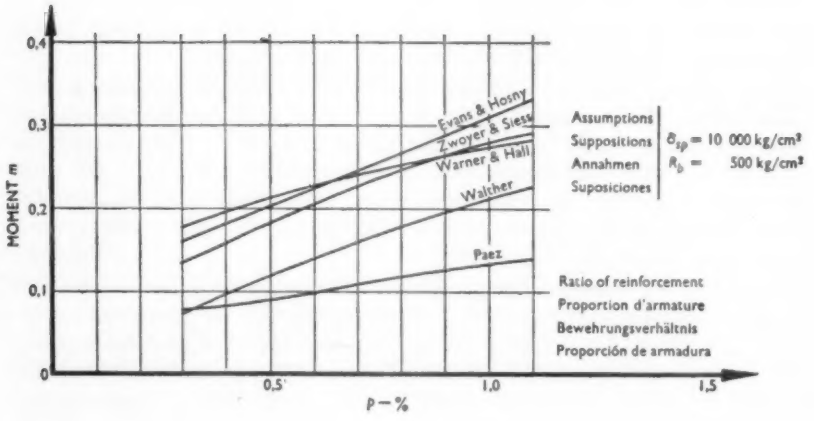

Figure 5

THE TORSIONAL STRENGTH

On this subject only one contribution was submitted, that by Cowan of Australia (No. 14). He conducted torsion experiments on post-tensioned beams with rectangular cross-sections. His work is very valuable because it deals with a field rarely touched before. The tensile stresses computed for the cracking load were in good agreement with the tensile strength of the concrete. In beams subjected to pure bending the resistance – as we know – is not exhausted when the

first cracks appear, but in beams subjected to pure torsion, ultimate failure immediately follows the formation of the first crack. The behaviour of beams subjected to combined bending and torsion is intermediate between these two limiting cases.

SHELLS

Prestressing offers a valuable chance to reduce the bending stresses caused by edge interferences in shells. Three interesting papers on this problem were submitted.

Svoboda of Czechoslovakia (No. 15) is engaged with the edge interferences that occur in a post-tensioned cylindrical container wall owing to the monolithic connexion with the bottom. He describes the suggestions put forward by different authors in their attempts to reduce the bending moments, and then further develops, on the basis of a theory by Havelka, the formulae necessary for an exact analysis of the stresses in the bottom slab.

Eimer of Poland (No. 16) studies the danger of buckling with post-tensioned cylindrical containers and comes to the conclusion that instability of the shell can only occur after a small, forced deformation. The formulae given at the end of the paper show that any danger of buckling in containers built of concrete can only exist in extreme cases.

Barets of France (No. 17) has performed interesting experiments on a large-scale model of a prestressed shell while developing a simplified theory for the computation of the stresses in cylindrical shell vaults. These tests allow for a comparison between theory and reality.

PRESSURE TUNNELS

The prestressing of pressure tunnels can be carried out by injecting the space between the so-called core ring and the rock with mortar, under high pressure. Kastner of Austria (No. 18) presents an interesting paper, including the computation of the stresses developed in the core ring, in which he clarifies the very complicated relations between the elastic and plastic deformations of rock and concrete. He also illustrates the developed formulae by means of numerical examples.

BRIDGES

Voves of Czechoslovakia (No. 19) describes experiments that were performed on a prestressed bridge with a span of 32 m. He discusses the measured deflexions and deduces from them the lines of influence for the lateral distribution of the loads and for the parameters which define the deformation of the cross-section.

In the paper by Klimeš, also of Czechoslovakia (No. 20), experiments on two bridge girders of a span of 25 m are described. Unfortunately, as some necessary data are missing, a comparison of the submitted results of the theory is not possible.

GENERAL EXPERIENCES AND BUILDING CODES

Harris of Great Britain (No. 21) reports British experience in the field of prestressed concrete. He then discusses the different opinions on the question of whether designing should be based on the theory of elasticity and on permissible stresses, or on the theory of plasticity and ultimate load.

The paper by Gvozdev, Dmitriev, Kalatourov and Mikhailov of the U.S.S.R. (No. 22) contains a description of the building codes valid in Russia for the design of prestressed structures. It is especially interesting where it describes in

some detail advanced Russian opinions concerning problems of safety and conditions of serviceability. Unfortunately it is not possible to go into details within the compass of this report.

Paduart of Belgium (No. 23) takes as a basis for his paper the comparison of the specifications laid down in the various countries, which he presented at the Second Congress in Amsterdam. He makes a new proposal for an international unification of the specifications laid down for prestressed structures. He asks for critical comments, in order that the extent to which such a unification is already possible may be revealed. This proposal deserves general support.

A great number of the papers submitted show remarkable progress in some fields. The problem of prestressed concrete farthest from a solution is still shear strength but although a final solution has not yet been found, the existing experimental results and hypotheses of failure do permit us to design safely.

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THIRD CONGRESS OF THE
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BERLIN 1958

SESSION II GENERAL REPORT

Progress in site techniques

by G. F. Janssonius

[*Nous sommes en effet sur le chantier*
où la perfection n'est pas toujours la règle
G. DREUX (Paper No. 8)]

PAPERS DEALING WITH PROBLEMS encountered in prestressed concrete constructional practice have been received from a number of countries. Many of the authors have presented their views – as based on experience – in a systematic form, and each has done it in his own fashion. It is, perhaps, hardly surprising that the authors' professional environment and nationality are sometimes recognizable in their approach to the various problems and in the way in which they tackle them.

Similarly, the compiler of a summarizing report can hardly escape the influence of his professional and national surroundings. If, therefore, this report emphasizes certain points in a way rather different from that anticipated by authors of individual papers the reason for this must be sought in the above-mentioned fact.

GROUTING

Papers: No. 1 Boardman (South Africa)
No. 2 Champion and Hill (British National Report)
No. 3 Ipsen (Denmark)
No. 4 Leonhardt (Germany)
No. 5 Mühe (Germany)
No. 6 STUVO-CUR Committee on Grouting (Netherlands)

As pointed out by Champion and Hill the quality of grouting does not always conform to the anticipated service life of the structure. In the sphere of post-tensioned prestressed concrete the improvement of the quality of grouting is therefore still an urgent problem.

The STUVO paper calls attention to the main objects of grouting, as previously stated by Kelopuu at Amsterdam in 1955. They are:

to protect the prestressing steel against corrosion and thus to increase the durability of the structure;

to increase the ultimate strength by producing the best possible bond between the prestressing steel and the surrounding concrete.

Boardman considers it important that the shear resi ance should also be increased.

We are immediately faced with the question as to what harmful consequences are likely to arise from inadequate execution of the grouting and what experience of adverse effects has so far been gained. It is understar..lable, but nevertheless disappointing, that none of the papers gives an answer to this question. This must not, however, lead us to infer that there are no grounds for concern on that score; this will at once be evident from the contents of all the papers submitted. The STUVO paper does, indeed, describe frost damage, and Ipsen also considers damage due to this cause. But these are cases of damage which occur at the time of contruction and not of structural damage arising after some length of time.

The STUVO paper formulates the following five requirements which the grout must fulfil after hardening:

it must fill the ducts;

when set it must adhere closely to both the steel and the inside walls of the ducts;

its compressive strength must be adequate;

it must not, a few days after setting, swell at low temperatures;

it must not corrode the steel.

Champion and Hill express the view that the design of structures is not affected by grouting problems. This may be true of British practice, but when we contemplate the increased difficulties involved in the grouting of so-called 'concentrated tendons' in accordance with German practice (see Leonhardt with reference to the need for grouting the ducts a second time and the desirability of using standpipes) we must indeed consider the question whether it would not be advisable to achieve better adaptation of the design to the possibilities of correct grouting procedure.

Several authors mention the requirements that the fluid grout must fulfil. These have been formulated most completely by STUVO:

its fluidity must be adequate so that grouting can be carried out under considerable pressure;

its loss of fluidity during mixing and grouting must be small;

it must be stable;

it must be resistant to volumetric changes;

it must have adequate compressive strength;

it must be frost-proof some days after grouting;

slight variations in its composition must not bring about great fluctuations in its properties.

The conflicting nature of these requirements gives rise to the problems with which we are here concerned. The last requirement is particularly significant, though the STUVO paper does not, unfortunately, give any details of it.

The most serious mishaps encountered in practice are, perhaps, those arising from failure to comply properly with the last requirement but one. The fact that both Ipsen and Leonhardt deal very fully with this point, whereas Boardman and Champion and Hill do not discuss it, is possibly due to the different climatic conditions prevailing in the countries concerned.

The grouting procedure usually employed in various countries is as follows. To begin with, the duct is cleaned by flushing it with water. Leonhardt mentions the use of warm water in the event of ice having formed in the duct. Next, as a rule, the water is removed from the duct by blowing it out with compressed air. Champion and Hill imply that this latter operation may be omitted. In that case the water in the duct is displaced by the grout itself.

All the authors agree that wetting the duct has a favourable effect, both in a metal-sheathed duct and in an unlined duct formed direct in the concrete.

Some water will inevitably remain in the duct after flushing. This water will be absorbed by the grout. For this reason it is often recommended that grouting should be continued for some time after grout has emerged from the other end of the duct. It may be asked, however, whether the mess this makes is an acceptable consequence.

Numerous types of plant and equipment are available for mixing the grout. The apparatus described by Mühe appears to possess a number of attractive features. The British colloidal mixer, discussed by STUVO, likewise has favourable features. Leonhardt's fear of excessive heating of the grout seems unfounded.

Interesting details of apparatus for the testing of grout and of the results obtained therewith are given by Leonhardt and STUVO.

Champion and Hill envisage displacement of the water by the grout. In countries where frost constitutes a real danger this is a questionable procedure, however, for in the event of enclosure of water there will be a permanent risk of its freezing.

Boardman, Ipsen and Leonhardt call special attention to the importance of the type of cement used in the grout. Champion and Hill and STUVO do not discuss this point; STUVO's reason for this is probably that the Portland cement it envisages is the Dutch product, of which there is only one brand. It is reasonable to suppose that the divergence of opinion on the question of adding trass to grout is due to differences between various makes of cement. In Holland the addition of trass is common practice, whereas in Germany it is not allowed. This state of affairs would appear to justify a proposal that a comparative investigation as to the suitability of various brands of Portland cement for grouting purposes should be undertaken on an international scale.

In the various papers a number of different grout compositions are considered. The use of blastfurnace cement in winter is deprecated on account of its slowness in hardening. Boardman and Leonhardt warn against the harmful effect of calcium chloride on high-tensile steel.

Ipsen mentions the use of methylated spirit. With regard to Dutch practice the relevant report presented at the Second Congress at Amsterdam in 1955 may be consulted.*

* THE STUVO COMMITTEE ON GROUTING. Some facts about Grouting. *Second Congress of the Fédération Internationale de la Précontrainte, Amsterdam 1955.* London, Cement and Concrete Association, 1958. pp. 14–29.

The statements made by the various authors on the use of aluminium powder are of considerable interest. Leonhardt explains its action. The aluminium causes the alkalis of the cement to give off minute hydrogen bubbles, which produce an increase of volume of the grout. According to Boardman and Champion and Hill this effect would more or less compensate in advance for any shrinkage that might subsequently take place. The STUVO paper, however, shows this to be an erroneous view. It is pointed out in that paper that the tendency to expand, due to the presence of the aluminium powder, does not at the same time imply an increase in pressure where the process takes place in a closed space such as a prestressing cable duct. Dutch experience has shown that the expanding action of the aluminium powder ceases as soon as further expansion of the grout is prevented. The powder has no effect on the shrinkage of the grout. The Dutch view is that there is no point in using aluminium powder in closed ducts.

Finally, two special problems call for mention. Mühe refers to the use of emulsified cutting-oil for reducing the friction of the cable against the wall of the duct. He regards the removal of this oil (by flushing it away) as a problem. In his view the presence of corrugations on the wall of the duct provides good bond between the grout and the wall.

Boardman proposes adding a radioactive isotope to the water used for flushing the duct. It would then be possible to detect by external observation whether or not the water had been completely displaced by the grout. There are, however, two objections to this interesting suggestion. As has been mentioned above, the direct displacement of water by grout in countries where severe frosts are likely to occur must be deprecated. Furthermore, personnel who are expert enough to handle radioactive isotopes may, surely, be expected to carry out the normal grouting operations in accordance with the rules of good practice, and there should thus be no real need to apply the proposed radioactive check.

FRICTION

Papers: No. 7 Barredo (Spain)
No. 2 Champion and Hill (British National Report)
No. 8 Dreux (France)
No. 9 Fritz (Germany)
No. 10 Kalhauge (Denmark)
No. 11 Knesch (Germany)
No. 12 Kowalczyk and Zieliński (Poland)

Dreux rightly states in his paper that a site engineer should have a sound knowledge of the phenomenon of tendon friction, and he of course implies that a design engineer must also possess an accurate and comprehensive knowledge of this phenomenon. It is essential to analyse all possible causes of friction. It is not enough merely to say, as Knesch does, that the magnitude of the friction is dependent on the prestressing force.

There are, in the main, three places where friction may occur, viz. in the jack, possibly in the anchorage device (e.g. in the Freyssinet cone anchorage), and between the prestressing steel and the wall of the duct.

The friction within the jack is difficult to ascertain in practice. Even the rather drastic suggestion made by Kowalczyk and Zieliński that each jack should be individually calibrated in a laboratory must be regarded as fundamentally

unacceptable. The treatment of the jack on the site, its maintenance, and the inevitable presence of grit and dirt are unpredictable factors of more than negligible importance.

In addition, general experience has shown – and this is moreover confirmed by Dreux, Fritz and Kalhauge – that pressure gauges are instruments whose efficiency and accuracy must never be relied upon. Again the view expressed by Kowalcyzk and Zieliński would therefore seem somewhat optimistic. Hence there is a general tendency to regard measuring the tendon elongation as preferable to relying on readings of a pressure gauge. Indeed, it is rather surprising that now, in 1958, after so much practical experience has been gained in many countries, it should still be necessary to dwell on this point.

For the sake of completeness it should be noted that the present general reporter has obtained excellent results with a hollow pressure cell provided with electrical resistance strain gauges which is placed between the Freyssinet female cone and the jack. Experience extending over a period of five years and relating to a number of different jobs and site conditions suggests that measurements of this kind are preferable even to measuring the tendon elongation. The tendon elongation of course is also observed.

Dreux, and Kowalcyzk and Zieliński give information on the magnitude of the friction in a Freyssinet cone anchorage.

The friction between the prestressing steel and the wall of the duct is a quantitatively more important and qualitatively more complex phenomenon. Several authors mention the Euler-Cooley-Montagnon formula. As Kalhauge points out, this formula holds good for an infinitely thin tendon and is subject to the condition that the magnitude of the friction is proportional to the lateral pressure exerted by the tendon. Fritz's version of this formula is as follows:

$$V_a = V_s e^{-\mu_s \kappa \rho (\Sigma \alpha + \beta l)}$$

The following are some practical points mentioned in the various papers.

According to Fritz the friction of rolled steel is greater than that of drawn steel. Champion and Hill note that the friction against a concrete duct wall is greater than that against a steel-lined wall. The same report mentions the use of lead-coated sheaths and aluminium sheaths, both of which are new developments.

Fritz is of the opinion that larger corrugations in the sheath are somewhat more favourable than smaller ones.

A very important phenomenon which is too frequently overlooked is the so-called 'wobble' effect of cables, to which Dreux and Kalhauge call special attention. The small and – on cursory inspection – hardly noticeable deviations of the cable profile from a truly straight line are found to cause a not inconsiderable amount of friction. According to Fritz the lack of complete straightness of the cable sheaths may be caused by inaccuracies in installing them, by deflexion due to their own weight, and by lack of care in placing and vibrating the concrete. Kalhauge's statement that the relation between the dimensions of the cable and those of the duct is very important becomes significant when viewed in this light. A wide sheath produces less friction.

It should be noted the a 'wobble' effect which cannot be entirely controlled occurs, for instance, in the transverse prestressing of a bridge deck composed of precast beams. This applies, in particular, to skew bridges.

Finally, rusting of the steel constitutes a factor whose relation to friction is difficult to assess. Fritz's view that a very light coating of rust will probably wear off represents only a partial reduction of the existing uncertainty on this point.

A phenomenon referred to as 'squeezing', i.e. the displacement of individual wires in a bundle or cable, may considerably affect the magnitude of the friction.

Dreux describes an experience which he at first regarded as an anomaly. He found a non-linear relation between the elongation and the prestressing force, particularly at the start of tensioning. He believes that this phenomenon is due to frictional resistance caused by grout ('fat') which has leaked from the surrounding concrete into the cable sheath and has hardened. His statement that this effect occurred chiefly with cables which were difficult to insert in position lends weight to this explanation. Dreux also suggests the catching or sticking of tying wires as a separate cause of friction.

It stands to reason that attempts have been made to reduce friction by the use of lubricants. It is not hard to agree with Kalhauge that the lubricant should be soluble in water and should be easily removable by flushing the duct. According to Fritz the only advantage of lubricants is that they help to prevent rusting. In his opinion, however, this is outweighed by the disadvantage that dust and grit tend to cling to the sticky lubricant. In view of the foregoing points the statements made by Kowalczyk and Zieliński on the use of paraffin call for some clarification.

Making a virtue of necessity, as it were, Barredo develops a principle which is of interest as a hypothetical experiment. He makes use of the fact that the direction of the friction on release of the stress is opposite to its direction on tensioning the cable. By adopting a carefully planned programme of tensioning and releasing he seeks to achieve a.more or less uniform stress in the prestressing steel of sharply curved tendons.

By way of summarizing the foregoing, it is appropriate to sound a note of warning to the effect that theoretical considerations on the subject of friction must on no account lead us to conclude that these phenomena as a whole are amenable to exact calculation. The too precise figures given by Knesch could produce a misleading impression. The present general reporter considers it important that the engineer should so design his structure that all the factors producing friction will be reduced as far as possible. For the same reason it is essential to ensure very careful execution of the work. Even if these two conditions are satisfied, Kalhauge's valuable suggestion that a friction test should be made on the site is worth noting and applying.

Kowalczyk and Zieliński point out that underestimation of the friction in the design calculations will result in too low a factor of safety. Less obvious, but certainly not less appropriate, is their warning that over-estimation of the friction may lead to the formation of cracks.

The present general reporter is of the opinion that the only really satisfactory cables are those which turn through as small an angle as possible, and he strongly deprecates the use of cables which curve sinuously in two directions.

It is worth mentioning Dreux's advice on the possibility of the actual friction turning out to be rather different from that envisaged at the design stage; appropriate precautions should be taken to ensure that certain acceptable tolerances will at any rate not be exceeded.

Another point made by Kowalczyk and Zieliński is the following. An error in the estimation of the amount by which the actual prestress differs from the theoretical value may have more serious consequences in a structure containing a small number of cables than in one containing a large number. In this respect, for example, a factory building in which the beams often function as individual members may be at a disadvantage in comparison with a bridge consisting of interconnected and structurally co-operating beams.

ANCHORAGES

Papers: No. 13 Barets (France)
No. 2 Champion and Hill (British National Report)
No. 14 Courbon (France)
No. 5 Mühe (Germany)

The four above-mentioned papers all report on new types of anchorages.

Two new anchorage devices are discussed by Champion and Hill. These represent two extreme types.

On the one hand, an ingenious single-wire anchorage, which is effected by means of a split conical grip, is described. In relation to earlier anchorages of comparable design this method has the advantage that the little wedges cannot be dropped or lost.

On the other hand, the above report gives some information on cables 200 ft in length, as described by Parker, which are provided with a 'blind' anchorage at one end. A cable of this kind has a capacity of up to 1,400 tons and is used in the construction of large dams. Projects now in the planning stage envisage cables that can be stressed up to 2,500 tons each.

Courbon describes an anchorage device which is used in conjunction with a new type of prestressing cable. This 65-ton cable comprises seven strands, each of which is composed of seven wires of 3·6 mm diameter. The anchorage consists of a hollow barrel with seven internal wedges. On forcing the barrel over the wedges, the strands of the cable are pressed into the wedges (which undergo a certain amount of plastic deformation and thus take an imprint of the strands). The force exerted by the cable is taken up by the wedges and transmitted to the concrete through a bearing plate. This method requires a special double-acting jack. The two functions of the jack are tensioning the cable and effecting the anchorage.

Mühe describes an arrangement by which the ends of the wires are fixed with high-strength concrete in a tube one end of which is slightly flattened.

Barets gives a description of a method of forming an anchorage at the non-jacking end of a prestressing cable. It would appear advisable to investigate more closely the stresses in the prestressing steel and in the anchorage device itself.

STEEL AND GLASS FIBRE

Papers: No. 14 Courbon (France)
No. 15 Kajfasz (Poland)
No. 16 Levi (Italy)
No. 5 Mühe (Germany)
No. 17 Schmerber (Germany)

In view of the importance of Levi's paper we must overlook the fact that it really does not come within the scope of Session II. Referring back to the Second

Congress at Amsterdam in 1955, he reminds us that the durability of high-tensile steel is a complex notion. For this reason the Italian regulations do not give merely one characteristic value, but a series of margins within which the various properties must lie.

Levi gives some interesting information on relaxation tests extending over a number of years (up to a maximum of nine years). In many another country such tests so far exist only in the realm of wishful thinking.

Levi's paper does not yet give an answer to the burning question whether he has, perhaps, detected opposite trends in the results of short-term and long-term relaxation tests. His desire to test a great many more specimens appears justified. At the same time it must be borne in mind that the manufacturing processes employed by the various makers of prestressing steel – which processes are kept secret for commercial reasons – are continually being modified and improved. It thus remains an open question whether a large number of relaxation tests – each of which is a time-consuming operation – will ever lead to a definite final conclusion. The future trend may well be that designers will continue to allow for some measure of uncertainty in this respect.

A noteworthy feature of Schmerber's paper is that it contains some very interesting photographs of mechanical defects in high-tensile steel. The discussion of the nature of these defects, the causes that produce them, and the frequency with which they occur, forms a fitting counterpart to Levi's paper, which confines itself to physical properties.

A significant fact emerging from Schmerber's experience is that wires may fracture some considerable time – sometimes even several days – after being tensioned.

Schmerber mentions an electrical testing apparatus through which all wire is passed prior to being used. Practically every mechanical defect in the steel is brought to light by this means. The problem of assessing the significance of the defects thus detected remains a difficult one, however. Schmerber's list of causes of mechanical defects may be 'the writing on the wall' for those who are inclined to go on increasing the permissible stresses to higher and higher values, basing themselves on experience gained in the laboratory. The list is as follows:

impurities in the rolling material;
laminations, laps, and flaws produced in the rolling process;
flaws produced in the drawing process;
defects caused by excessive bending of the wires in the course of special manipulations;
damage due to improper storage;
damage caused during transport;
damage resulting from the use of wrong methods in uncoiling the wire.

Mention should finally be made of Schmerber's electrical tests on the steel during tensioning and in the completed structure.

As has already been said in the section on anchorages, Courbon's 65-ton cable consists of seven strands composed of seven wires of 3·6 mm diameter. On account of its flexibility this cable is easy to install. The high quality of steel associated with the use of these small wire diameters is a considerable advantage. Owing to the low apparent modulus of elasticity, the effect of shrinkage and creep of the concrete is less than with other types of cable.

The sheath used with this flexible cable is a rigid, thin-walled steel tube. The wall of this tube is smooth, i.e. it differs from many other types of sheathing in having no ribs or corrugations. The question arises as to what influence this smooth wall has on friction. Furthermore, the favourable effect of corrugations – which, in the event of segregation of the grout, serve to keep some of the foam and segregated water away from the prestressing steel – is lacking in this case. Greasing the cables would appear to be a permissible procedure, since the twisted strands ensure good bond. It is, however, essential to remove all traces of grease from the smooth internal wall of the sheath.

Mühe suggests the possibility of making small cables in a manner similar to that employed in the manufacture of electric cables. It is not indicated in his paper whether this original suggestion is being carried into effect.

Those who are interested in the properties of glass fibre will find a fairly comprehensive treatment of the subject in Kajfasz's paper. As no other papers dealing with this material have been submitted and as the present general reporter has no views of his own on the merits of glass fibre, this mere mention of the paper must suffice.

MISCELLANEOUS

Papers: No. 18 Lessing (Germany)
No. 19 Vassiliev, Courec and Perelstein (U.S.S.R.)

Both the above-mentioned papers are really on the border-line between Session II and Session IV. It is not intended to prepare a General Report or to print any papers on the latter theme. Therefore only a brief indication of the subjects dealt with in these two papers will be given.

Lessing describes the construction of the Nahe viaduct near Bingen (Germany). It has seven spans and an overall length of 367·50 m. The author discusses the choice of the positions of the construction joints in relation to the location and magnitude of the prestress, which is produced by means of Freyssinet cables consisting of twelve wires of 8 mm diameter.

Vassiliev, Courec and Perelstein describe the prefabrication and erection of large numbers of beams for the roofs of industrial buildings. Beams up to 18 m in length are assembled from units 3 to 6 m long which are cast in a horizontal position. The desired shape of the unit is produced by the mould and by means of a heavy ram. To cater for different working loads the number of cables and the quality of the concrete, but not the shape of the units, are varied. The paper also deals with lattice girders up to 30 m in length. In these girders the bottom chord and sometimes also the end diagonals are prestressed.

There are two further points of interest in this paper, though detailed information on them is, unfortunately, lacking. The constructional transition from a cable to a bar with a threaded end and nut is worth noting. The use of site-welded steel plates for forming the connexions between concrete units would appear to be another new feature. The prestressing steel is connected to these plates.

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SESSION III GENERAL REPORT

Progress in the manufacture of factory-made precast prestressed concrete units and in their use and assembly on the site

by D. H. New

CONTRIBUTIONS ACCEPTED for publication were as follows: National Reports from the Netherlands and Great Britain, and individual papers from Czechoslovakia (2), East Germany, West Germany (2), Great Britain (2), New Zealand, Poland, Rumania, Spain, Switzerland, U.S.A. (3), and U.S.S.R. (2).

As for the Second Congress there were notable omissions by countries that have done outstanding work, especially in the field of factory production.

It is regrettable that, owing to lack of space, some contributions containing information of interest could not be accepted, while others had to be considerably reduced. A general system of national reports might well have eased this situation.

FACTORY PROCESSES

An outstanding contribution is that of Berditchevski, Svetov and Sklyar of the U.S.S.R. (No. 1) which describes the large-scale factory production of prestressed units by the machine application of a continuous prestressing wire. Members of the Second Congress will remember a film of some of this work.

Floor slabs are constructed on a rotating table, the stressing wires being passed under tension round a series of steel pins accurately located in the table, and protruding into the unit. Because of the many turns taken round these steel pins, a very high factor against failure due to lack of bond is claimed.

Beams are normally produced in pairs side by side, the continuous wire passing

down the length of one beam, round semicircular end cheeks, and then into the other beam of the pair. The wires round the end cheeks are finally cut away leaving two quite normal fully bonded beams. The process reminds one of the Leonhardt process on a small scale, with the exception that the wire is stressed during application instead of after.

Wall units are formed of a series of prestressed stiffeners passing through one another at right-angles and integral with a normal reinforced concrete facing slab 2·5 cm thick, reinforced with welded steel mesh.

Roof beams are made up of a series of two or three standard concrete units, with integral steel connecting pieces which enable tension in the bottom flange to be transmitted through welded or bolted connexions. The continuous prestressing wire in these standard units is so positioned that it not only resists bending but also shear, especially near the supports.

It is pointed out that the methods described are particularly suitable where a factory, or section thereof, can be exclusively devoted to the long-term production of a particular unit, which means the standardization of building on an immense scale. While most members of the Congress are unlikely to meet this degree of standardization in their own work, the approach to the problem may well stimulate thought in other directions.

The British report (No. 2) discusses comments received from about thirty firms in reply to a questionnaire circulated for the purpose of obtaining details of general practice. This indicated that the most common type of wire used for fully bonded work is 5 mm, closely followed by 7 mm. A maximum size for screwed rods of 28 mm is mentioned. It is apparent that stranded cables are not normally used.

The use of deformed wire is much more widespread than of crimped. (If, however, the work in Base's paper (No. 9) is followed up it may well be that crimped wire will steadily gain in favour.)

The maximum weight and length of units produced by the majority of firms is generally from 5 to 10 tons and 18 m, the highest quoted figures being 34 tons and 33 m.

Steam curing is not reported as normally being used on the long-line process, and only one firm used Ciment Fondu for other than extremely urgent work. Some figures for 'release stresses' are given and reference is made to the draft Code of Practice.

There appears to be some swing away from the use of I-section beams to rectangular hollow beams, and it would be useful to have a discussion on the economics of this matter.

As mentioned earlier, it is unfortunate that lack of space did not allow the publication of all papers submitted. One of these, by Simonnet of Mexico deals with the production of small beam units on the long-line process. The novel feature is that the forms, which are in heavy steel, are incorporated into a travelling machine that works on rails running the full length of the bed. The machine is brought into position, the forms lowered on to the bed, and the very stiff concrete placed round the stressed wires under conditions of intense vibration. The operation of placing and vibrating the concrete being complete, the machine operates immediately to demould, by drawing the moulds upwards, with an initial slow movement followed by a quick release, and then passes to its position on the next beam. The method would presumably not lend itself to use with all

types of aggregates, but, if it could be used and applied to repetition work of sufficient quantity, it should lead to economies.

Another paper that could not be published was by Seltenhammer of Austria; this describes an unusual procedure for radially prestressing manhole covers. The circular concrete slab units are matured for a much longer period than usual. A steel ring accurately made to size is heated to a red heat, placed in position round the unit, and immediately quenched before the concrete can be affected by the intense heat. Some details of tests under heavy blows are given. It is not clear what prestress is obtained nor what losses are envisaged with time.

The paper by Price of the U.S.A. (No. 3) deals mainly with the production of standard units on the fully bonded long-line process, particularly double-T units, prestressed with 7-wire strands. The demand for these units has increased so rapidly that now nearly two hundred factories are engaged in their manufacture. Mention is made of the difficulty in controlling differential camber between similar units, a matter which is of course of considerable concern to all manufacturers. It states that in order to reduce the number of skilled men needed there is a move in the interest of economy for post-tensioning generally to replace the long-line process. This has not been suggested from any other country.

Gerwick of the U.S.A. (No. 4) makes the interesting point that, in view of the big handling problem involved in moving heavy bridge beams in the factory, it has been found more economical to manufacture a number of single beams side by side on a short wide bed than to use the long-line process. The practice of deflecting a proportion of the fully bonded tendons upwards towards the ends of beams by the use of suitable anchoring devices is discussed.

Mention is made of accelerated curing by placing well built insulated steam hoods over the units, which was found to give much improved results when compared with curing under canvas or tarpaulins.

Smith of the U.S.A. (No. 5) discusses the growth of prestressing in the United States and the advantages of precast concrete as compared with cast *in situ* construction. Details are given of a typical plant for producing units by the long-line process.

Vondráček of Czechoslovakia (No. 6) describes the manufacture of concrete pipes, prestressed longitudinally with a series of single straight wires and radially with a single continuous wire. Maturing time is reduced by heat treatment.

Staněk of Czechoslovakia (No. 7) emphasizes the economic advantages of setting up a factory adjacent to the site for the production of large beams instead of having the beams delivered from a central depot.

Kirchknopf of West Germany (No. 8) described the fabrication of beams of up to 30 m span on site, prestressed by an end-anchored method, making the point that this avoids transport from a central depot.

DEVELOPMENT OF BOND

This matter is of fundamental importance to all manufacturers of fully bonded prestressed elements, and the papers by Base of Great Britain (No. 9) and Ratz, Holmjanski and Kolner of the U.S.S.R. (No. 10) are worthy of close study.

From the point of view of the manufacturer the work by Base is of particular interest because 60% of the tests mentioned were carried out in factories under normal working conditions and the paper refers to a mechanical strain gauge

well suited to commercial work. The other paper bases its results much more on theory combined with laboratory tests, but nevertheless brings out some quite interesting points.

Both papers show that bond is affected to a high degree by the strength of the concrete, a point which should be borne in mind by those who seek to prestress concretes of relatively low strength. In this respect the work by Base shows longer transmission lengths for wires in the top of a beam than for those in the bottom, clearly indicating the difference in consolidation that is normally obtained.

The Russian paper finds increase in bond with increase in depth of indentation, but the depth of these indentations as regards fatigue appears to need investigation. The crimping of wire showed good results.

Wires placed in pairs do not appear to suffer loss of bond as compared with single wires, and tendons made up from twisted wires show considerable promise, especially when compared with plain wires in concrete of low strength. Base suggests that deformed 16 mm diameter screwed bars are suitable for fully bonded work, provided that a transmission length of 75 cm is acceptable.

Both papers give details of how the stress builds up in the anchorage zone, and bond lengths for a range of wires. With smaller diameters there is a large local peak strain, very close to the end of the beam, well before full stress distribution is obtained.

The effect of time on bond does not appear to be serious but tends to be more for plain than for indented wire. Base indicates a serious increase in transmission length where wires are released suddenly, and draws attention to the need for proper compaction at the ends of units to ensure high-grade concrete round the wires in the region of bond build-up, a matter of which the importance cannot be over-emphasized.

Base gives some measurements of tension at the ends of a beam in a vertical direction and shows that the stirrups to resist this should be concentrated near the ends with a minimum of spreading out. Gerwick of the U.S.A. (No. 4) also makes some mention of the incorporation of vertical stirrups to prevent the development of longitudinal cracks, but suggests that the stirrups should be carried on for some distance along the end block.

BRIDGE BEAMS

The papers submitted lay considerable emphasis on bridge beams, thus indicating a big field of future expansion for prestressed concrete, since obviously many countries will be improving their road and rail communications during the next few years.

Sutherland of New Zealand (No. 11) indicates how the unwillingness of local contractors to put in competitive tenders for prestressed bridges was overcome, and the ideas given might well be useful for engineers who attempt to introduce the technique of prestressing under similar circumstances. A series of standard bridge beams was evolved made up of units weighing between 3 and 6 tons each, stressed together on site with 2·5 cm wide joints between, either poured or packed with aluminous cement mortar, and stressed within twelve hours. The assembled bridge beams were normally stressed together transversely.

Gerwick of the U.S.A. (No. 4) deals with the construction of bridge girders up to some 85 tons in weight and 36 m in length, prestressed by the fully bonded

method. Some details of the transport of long and heavy girders are given, including ocean tow on barges.

Price of the U.S.A. (No. 3) gives details of hollow bridge beams and mentions, as does Gerwick, the deflexion upwards at the ends of tendons in fully bonded beams. He states that post-tensioned beams are prestressed with screwed rods, Freyssinet cables, or wires with button heads.

Abeles of Great Britain (No. 12) deals with the standardization of bridge beams on British Railways, Eastern Region. For short spans a series of inverted T-beams has been standardized; on these, normal reinforced concrete is cast at site, to give composite construction. Partial prestressing has been used in these inverted T-beams, a maximum working tensile stress of 45 kg/cm^2 being allowed except for under-bridges (where the tensile stress must be zero). For spans of over 18 m, hollow box-shaped precast beams are preferred.

Where very shallow construction is considered to be essential, even at increased cost, reduced depth is obtained by what is referred to as 'wafer-slab' construction, in which the prestressing and concrete placing is carried out in two stages. In the first stage the concrete is stressed up to 210 kg/cm^2 and in the second stage, using partly bent-up cables, it is brought back to this figure to make up for the losses that will have occurred from creep and shrinkage.

An Appendix gives details of a loading test which was carried out on a 'wafer-slab' bridge, at Cooks Green Lane, near Clacton, Essex. Attention is drawn to the very satisfactory distribution of load between the beams although cross stressing was not used.

Staněk of Czechoslovakia (No. 7) deals with the fabrication of bridge beams of up to 49 m in length, those above 25 m being made up of precast sections prestressed together. Some notes are given of the main prestressing systems, Baraba and Horel, used in Czechoslovakia. Erection of bridge beams with the assistance of Bailey bridging is described.

The British report (No. 2) mentions the footbridge to Eel Pie Island, Twickenham, with its two arched beams of 49 m span, made up of precast sections, assembled and stressed in their final positions.

The STUVO report from the Netherlands (No. 13) deals particularly with methods of producing continuity over supports. It points out that the use of normal reinforcing steel for this purpose is unsatisfactory because the shrinkage and creep in the adjacent beams builds up tension over the supports with consequent cracking. The cracking can, it is suggested, be reduced by bevelling the beams off at the top, widening them at the bottom, and then reinforcing with mild steel as before. While creep effects may well be less, this latter method appears to have the same inherent disadvantages as the first.

The cantilever and central span type of construction, used for the Amstel Bridge, and mentioned at the Second Congress, is briefly discussed.

Mention is made of a bridge at Klaprozen where a series of freely supported beams each 20 m long form continuous construction for a length of 100 m. Grooves are cast in the tops of these beams, and continuity obtained by stressing straight cables 100 m long running in the grooves.

The ferry boat jetty at Amsterdam, which was seen by members of the Second Congress, obtains continuity over the supports by stressing up short cap tendons through risings at the ends of the beams.

Lastly the report describes methods of obtaining continuity by means of

transverse cables in the region of the supports. In the first method, the beams pass one another across the points of support, joints being made between them longitudinally and transverse prestressing then being applied across these joints. The friction set up causes the beam complex to act as a monolithic whole and gives true continuity. A modification of the method is to place slabs, prestressed longitudinally to the bridge, between the top flanges of adjacent abutting beam ends and equally spaced over the supports. Transverse prestressing through the slabs and tops of the beams secures them together, thus constraining them to act monolithically. A third method closely follows the cantilever and central slab plan, but full continuity is obtained at the joint by cross stressing between the ends of the adjacent beams and a series of precast units.

PILES

The British report (No. 2) points out that the prestressing of hollow piles extends the range of length in which a given-sized pile can be driven with a monkey of given weight. It emphasizes the need for extra care in consolidation and the need for truly square ends if the pile is to stand up to heavy driving. An example is given of hollow piles for Beckton Gas Works 23 m long and 50 cm across an octagonal section.

It also describes a prestressing bed 132 m long laid down in the open at Fawley Refinery for the production of 25×25 cm piles, 11–14 m in length. This was later adapted to the production of circular hollow piles 45–55 cm in diameter, 20–27 m long, the cores of which were formed with light-gauge sheet metal.

Price of the U.S.A. (No. 3) also mentions prestressed piling of hollow section, the holes being either formed with cylindrical fibre-board tubes that remain in the concrete, or by inflated rubber tubes. He states that seven-strand tendons are invariably used for prestressing fully bonded piles. Post-tensioned piles of large diameter are made up from precast concrete cylinders prestressed together. No details are given of the type of joint used between the adjacent cylinders, or of the driving or sinking conditions, information on both of which would be of great interest.

THE ASSEMBLY BY PRESTRESSING ON SITE OF PRECAST UNITS

This has been dealt with, in some measure, under the subsection dealing with bridge beams.

The STUVO report (No. 13) commences with a comparison between this method and that of building in complete structural units. It calls attention to the need for the proper curing of dry-packed joints, an important point that can easily be missed. To deal with poorly made joints, and the cracking they might cause, it is suggested that the front surfaces of the elements at any joint should be reinforced. This would be expensive and there does not appear to be any particular reason why joints should not be formed satisfactorily. Comments from others would be of interest.

Some buildings have now been constructed as statically indeterminate structures, completely built up from a series of quite small members prestressed together on site. Examples are the garage at s'Hertogenbosch, where the columns had to be guyed while the cross members were placed and prestressed to them, and the paper mills at Wormer. This last example is quite outstanding, since the whole skeleton frame consisting of columns, gable, crane, roof and wall beams,

and roof and ceiling units, is all buit up of quite small precast concrete blocks, assembled on site to give a monolithic whole when prestressed. Generally there is no mild steel in the individual units, except for brackets and under anchorages.

Erection of buildings on these lines can, of course, be carried out with quite light equipment, but there are disadvantages in taking so much work to the site, and it might often be better to use heavier equipment and build up large units in the factory. Careful consideration should be given to matters of economy before these added site complications are undertaken; it is hoped that further views will be expressed.

Roš of Switzerland, in collaboration with Lax and Donner of Finland (No. 14), describes the prestressed concrete frames for the boiler house and turbine hall of a power station in Finland, the beams and columns for which were built up of a series of elements 2 m long with 2 cm wide joints between. The stressing of cables in pairs as the columns are built up is described together with a method for carrying the beam units before stressing by means of trussed timber beams.

The British report (No. 2) states that the dry-packed mortar joint about 2·5 cm wide is the most widely used method of jointing, and places considerable emphasis on the need for efficient caulking. It draws attention to the fact that 'buttered' joints are unsatisfactory, as they give rise to high local stresses with attendant cracking. Joints cast *in situ* are recommended where dry packing presents unusual difficulties.

The suggestion is made that dry-packed joints formed with Ciment Fondu are not satisfactory, which is quite possible because curing of the heart of the joint is likely to be very poor; comments would be valuable.

Attention is called to the need for precision in the positioning of cable holes, to enable the cables to be threaded easily through the junctions of adjacent units.

Some examples are given, outstanding among which is the hangar at Gatwick Airport with its series of 32 m span space frames stressed together transversely through the purlins connecting their top booms and resting on lattice beams of 43 m span across the main doors.

The factory at Southampton is a good example of the application of cantilever and centre span construction to building work. The photograph showing the centre span of a main beam, with its cantilever diaphragms to support the central spans of the secondary beams, is of great interest.

The 31 m span beams at East Kilbride follow the type of construction for the London Airport hangars described at the Second Congress, but they are designed for a heavier loading and have webs 12·5 cm thick instead of 10.

Barredo of Spain (No. 15) deals with prestressing together a series of precast units to form roof beams. The individual units, which are 1·53 m long, have cable holes in them formed by steel bars which are removed just after the concrete has set, but, unlike the units for the beams at East Kilbride and London Airport, the holes are always relatively in the same position, so that the cables run in straight lines from end to end. Proper distribution of prestress is obtained by varying the precast concrete units themselves, by making them deeper and wider at the centre than at the ends. This method of design, while leading to certain simplifications, eliminates the favourable shear condition that obtains with the use of bent-up cables, because there is only the horizontal prestress to reduce the tensile stress effects of shear.

Halmagiu, Scripcă and Kessler of Rumania (No. 16) describe in detail the

fabrication of concrete trusses by prestressing together a series of precast units, and give interesting examples of this type of work.

A paper by Bychawski and Eimer of Poland, that unfortunately could not be published, suggests that prestressing should be applied to joints in which the filling material is still plastic, and develops ideas on stress distribution based on the theory of viscous liquids. Steel-driven dry-packed joints between adjacent units prestressed together are now well established and appear to give complete satisfaction, but it might be useful to hear views regarding the possible advantages of this quite different approach to the jointing problem.

COMPOSITE CONSTRUCTION

Little positive mention has been made of the composite action between precast prestressed and normal reinforced concrete, mainly, in the reporter's opinion, because it is now in such general use as to be looked upon as commonplace.

The British report (No. 2), however, gives a few comments, and suggests that roughened or cross-tamped surfaces for the transmission of horizontal shear are being used to an increasing extent. Castellated surfaces are still mentioned, but, since they are often unsatisfactory in practice, comments regarding their continued employment would be useful.

Some notes are given on the approach to the problem of shear, a matter on which we should all like much more information. Reduction of prestress from differential shrinkage of the *in situ* concrete is mentioned, the tendency being to recognize that there is an effect but to neglect it in practical design.

Composite construction was adopted for the decks of the North Viaduct, Clifton Bridge, Nottingham. Abeles of Great Britain (No. 12) depends on composite action for the construction of 'wafer slabs' and inverted T-beam bridges.

GENERAL

The STUVO report (No. 13) mentions a hall for N. V. Philips of Acht, the units for which were obviously designed for close tolerances according to the clearances allowed in the dowelled connexions at the ends. Interesting points are that the dowel holes were finally run in with bitumen, thus allowing a measure of movement on the bearing, and the packing under the bearing is of sheet lead. A comparison between the use of sheet lead and mortar packing for this purpose would be of great interest.

Zieliński of Poland (No. 17) explains that the normal prestressed beam is only about 50% efficient as regards the proper use of material; he then deals with the development of a tied arch form of construction, which has been widely used in Poland to support roofs over spans of from 15 to 50 m.

Bührer of West Germany (No. 18) describes a method of constructing shell roofs by assembling a series of precast concrete units on appropriate struts and bearers, making the joints, and then forming the whole assembly into a monolithic shell roof by prestressing.

Mlosch of East Germany (No. 19) describes a system of prestressed concrete roof members, and methods of forming the joints between them.

CONCLUSION

The various submissions indicate a satisfactory growth in the use of prestressed concrete since the last Congress. Generally the advances seem to be in

the increase of size and weight and in the elaboration of existing methods rather than in the development of entirely new techniques.

This is to be expected because there are many problems (e.g. the development of bond, vertical stresses in the ends of a beam, joint conditions between adjacent elements, formation of economical and satisfactory bearings, the elimination of differential camber, and continuity over supports) on which there is much development work still to be done before spectacular advances in other directions are likely to be made.

There must of course be many items all over the world which although small in themselves would have been of great interest to members of the Congress. These various small but valuable items of information will only become generally available if a system of nation reports is adopted.

The reporter has advocated this for many years and would make a special plea that it should be the rule for the next Congress; he hopes that it would eliminate the necessity for rejection of some papers and the severe editing of others, which can be a very heavy burden on those responsible.

Our object, surely, is to make available all useful knowledge in a precise and easily assimilable form, and it is the normal duty of each one of us to do his best to achieve that end.

The reporter has drawn attention to certain items which he feels are of particular interest, especially to items of interest to those engaged in the manufacture of precast units, and hopes that contributors to the discussion will be able to give useful information on at least some of these.

The foregoing General Reports from Sessions I, II, III, Third Congress, Federation Internationale de la Precontrainte, Berlin, 1958, and the complete list of papers, are part of the F. I. P. Literature Exchange Program, and are being reprinted through the kindness of F. I. P.

Prestressed concrete design and production would not have attained its phenomenal rate of growth annually without the excellent assistance continually being given by the Portland Cement Association.
Their new laboratory facilities, their accomplished technical staff, and — above all — their close and cordial cooperation with PCI, promises new horizons ahead for prestressed concrete. As you read Dr. Bates' illuminating philosophy, you will at once share with us our admiration for them and their efforts towards our common objectives. *(Ed.)*

* PCA FIRE TEST CENTER

WORKS EXCLUSIVELY ON PROBLEMS RELATING TO PRESTRESSED CONCRETE

We are very pleased that a number of you are coming out to the Portland Cement Association Laboratory this afternoon, and I want to tell you a little about that because receiving several hundred men at that laboratory and trying to make the most of the occasion in their interest is not an easy task. You're coming to see the physical facilities of that laboratory, but I believe it is equally, or, in some ways possibly more important that you realize something other than the physical side of that installation. In other words, I think it's very important that you know something about the attitudes, the principles, the philosophies which guide and shape our work out there. The portland cement industry is in a very peculiar position, almost a unique position. It makes a product and has little or nothing to do with the use of that product. Portland cement is made, sent out to the world, a child of the industry and the industry knows little or nothing about what may happen to it when you, the users, put it to work. Now, the Portland Cement Association proclaims on its letterhead, in its constitution and its charter and by-laws, that the Portland Cement Association itself is "an organization to improve and extend the uses of portland cement." And everything we do at PCA is done with that ultimately in mind.

Specifically the research and development laboratory to which you are coming this afternoon is a laboratory to "improve and extend the uses of cement." We have three departments out there in the research and development division. One, the manufacturing process department, studies problems encountered in the production of cement. That department, although it is one of the three, spends only about 7 per cent of our budget. The other 93 per cent is expended on improving the uses of cement.

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*Remarks of Dr. A. Allan Bates, vice president for research and development, Portland Cement Association, delivered preliminary to the visit of members of the Prestressed Concrete Institute to the Portland Cement Association's Research and Development Laboratories at Skokie, Illinois, on September 24, 1958.

If there has been a spectacular extension and improvement in the use of cement recently, it is the use which is best represented right here in this room. It is the use in prestressed, precast products. And, if there is any group in the world today that we appreciate and love — it is YOU — right at the moment. And our work at the laboratory is done for you. Now I want you to feel that this is more than just a nice, oily statement. It is genuine. The uses of cement are going to be extended in more ways than those represented by you. But, at the moment, there is nothing that interests us as much as your problems and when you come out there, we want you to look around with the constant question in your minds, "What do they have here that can work on some of my problems?" Now, let me hasten to add — we are not a trouble-shooting outfit. This is a true research and development laboratory. We are not an invention mill. We are not trying to compete with you in doing business. We are trying to help you conduct your business to the best possible advantage and profit. As a true research laboratory, our views are long-range views. We are investigating long-range, fundamental problems which today are inherent in your work, but possibly less visible, less urgent to some of you than they might be.

When you come to the laboratories, you'll have only a bird's-eye, only a general view. You can't possibly see much in detail. And, in order that I may not forget — let me urge now that all of you return as often as possible, individually or in groups and with sufficient time available so that you and we can sit down together and discuss your problems. We may or may not undertake to attack any given problem you present to us. We will undertake problems only of general character. We will not undertake the solution of problems specific to any one manufacturer. Even a cement manufacturer cannot come to us and ask us to solve his problems. So we are a true research laboratory.

The Fire Research Center is not yet actually completed, but it is completed enough so that you will be able to get a concept of what is going to go on in there, and I am happy to tell you that for the next year we are going to be working solely on the problems of fire resistance of prestressed concrete. Prestressed concrete preempts that building solely for the next year. We have, ultimately, in prospect four major furnaces to be erected in there. The first of these will be available within a few weeks. That is a beam furnace, capable of taking a 60-foot prestressed beam. We have already under conditioning and curing, some 20 or 30 prestressed beams, varying in design. Within a month those beams will start through the beam furnace which will be ready for them. So, in that Fire Research Laboratory you will see nothing going on, but you will see a facility which is unique in the world and which for the next year or possibly several years, is going to be devoted entirely to your problems which are our problems also.

The Structural Development laboratory was completed about six or eight weeks ago. It is in use, but not quite total use. There are a number of operations going on of great interest to you and every operation in there today pertains to prestressed concrete. We are spending research and development money on prestressed concrete there at a rate which I know you'll be able to measure just by looking at the magnitudes of what we are doing.

DETAILS OF STEEL COLLARS

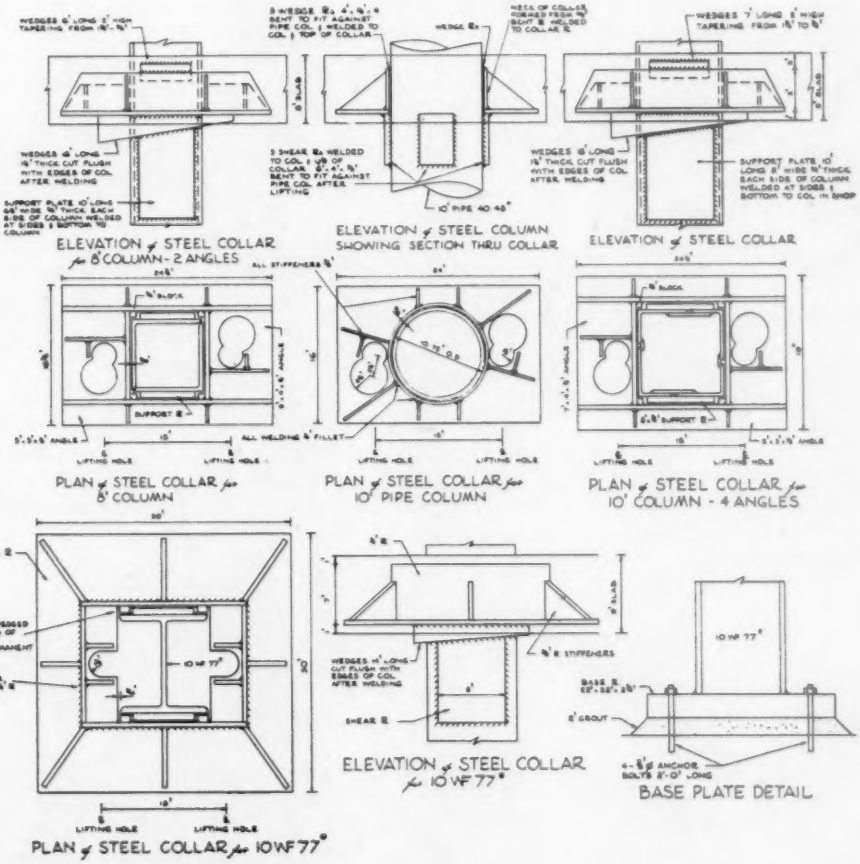
FOR LIFT SLAB CONSTRUCTION

From an article — "LIFT SLAB DESIGN AND CONSTRUCTION"

by

W. SEFTON, B.Sc.(Eng.), A.M.I.Struct.E., A.M.I.C.E.

Presented at the 50th Anniversary Conference, The Institution of Structural Engineers, October 10, 1958, London, England.


ELEVATION of STEEL COLLAR for 8' COLUMN - 2 ANGLES
ELEVATION of STEEL COLUMN SHOWING SECTION THRU COLLAR
ELEVATION of STEEL COLLAR
PLAN of STEEL COLLAR for 8' COLUMN
PLAN of STEEL COLLAR for 10' PIPE COLUMN
PLAN of STEEL COLLAR for 10' COLUMN - 4 ANGLES
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ELEVATION of STEEL COLLAR for 10 WF 77
BASE PLATE DETAIL

Summary of Technical Program presented at the FOURTH ANNUAL PCI CONVENTION Chicago, Illinois, September 1958*

By

Hubert C. Persons, Consultant, Chicago, Illinois

The fourth annual convention of the Prestressed Concrete Institute not only gave plenty of evidence that the industry it represents has grown up, but that it has also reached the hour of decision in many ways. For one thing, PCI members appear to feel that it is time to separate the exploiters and opportunists from the developers and innovators. The seriousness with which members discussed the proposed certification of prestressing plants is tangible evidence of this.

The make-up of the four-day program showed that PCI members are changing from being maker-minded to being market-minded as was urged by the keynote speaker, Charles Luckman. At least half of the program time was occupied with talks or panel discussions on marketing, product development or advertising. There was also a broader participation by members in the program activities at this convention than at any previous PCI meetings. This was largely due to the seven panel discussions in which 44 members were active participants in addition to a considerable number of members who enlivened the discussions with questions from the floor. Although the panel discussions included technical subjects, they were in no sense theoretical. Instead they were pertinent accounts of actual experiences of prestressed concrete manufacturers and users in plants and in the field.

Although Mr. Luckman was the official keynote speaker, an unofficial but significant keynote was sounded from the floor by a concrete producer from Utah. He declared that although he had attended many conventions of trade associations he had never been to one "that got down to earth like you are doing here today."

There was some emphasis on the fact that prestressed concrete is a new material unknown outside of the building industry. The consensus was that this fact made PCI's advertising and sales promotion job more difficult. One speaker said he had, at different times, mentioned prestressed concrete to 35 industrial executives each holding a policy-making position, and learned that not one of them had ever heard of the product. That might be a serious handicap to promotion were it not realized that while prestressed concrete itself is a new and previously unknown material, concrete and various products made from it have come to be synonymous with strength and durability in structures and in pavement. This is due in a large measure to years of work by the Portland Cement Association and others. So although prestressed concrete as an individual product may not be well known, the fact that it belongs to a rugged and widely used family of products, provides a firm basis for the education of potential users.

Most PCI members find it heartening to learn that a number of manufacturers of prestressing steel, as well as the Portland Cement Association have been helping to promote

*The fully transcribed technical program will be available from PCI at $5.00 the copy.

prestressing and will continue to push prestressed concrete in advertising and promotion literature. In remarks to the PCI convention delegates prior to their visit to the PCA laboratories, Sept. 24, Dr. A. Allan Bates, PCA's vice president for research, said that the fire test center and other new facilities at the laboratory would be working almost exclusively on problems relating to prestressed concrete for the next year or more.

Jackson Taylor, New York advertising executive, told the PCI members that development of a broad advertising and marketing campaign is the next logical step for their industry.

One of the matters of greatest interest discussed during the convention was the proposed PCI certification of prestressing plants. J. Ashton Gray, chairman of the Plant Standards committee, declared that self-regulation of the prestressing industry is "a must." Other highlights of the program were the assertion of a noted bridge engineer that spans 2,000 ft. long may now be designed in prestressed concrete; indications that draped strands will be generally adopted in future prestressed concrete fabrication, and predictions that use of prestressed concrete in the nation's big highway program could conceivably double the field of use of this material.

"The Sky Is the Limit," was the title of the keynote address by Mr. Luckman, formerly president of Lever Bros., but now a member of the architectural firm of Periera & Luckman, Los Angeles.

"The future is fantastic in its potential for the use of prestressed concrete design. The sky's the limit," the speaker declared. He suggested that prestressed concrete should be thought of as a new material rather than a new use for ordinary reinforced concrete.

"It is exciting to examine some of the uses of this new material," Mr. Luckman said. As one example he referred to a high bridge with 1,300 spans, built in the Lake Maracaibo region of Venezuela. He enumerated the following examples of promising uses, trends and developments:

"Standardization of prestressed products. Architects will accept, really want, standard dimensional products, design and costs.

"Utility of crack-free, deflectionless structures in architectural design. Also advantages of prestressing where frames must resist lateral loads of earthquakes and high winds.

"Uses in folded plate structures and cantilever shell structures.

"Architects need to know more about possibilities of repeating girder sizes and members where on a production basis, the cost can be very competitive.

"Railroad ties, transmission line poles, barges, overlays for airports.

"The United States is embarking on the largest program of public works ever conceived. In the next 12 years, 100 billion dollars of highway construction, bridges, bridges, bridges.

"But the greatest of all is use in buildings where prestressed floors, panels and spandrels and free form shells will revolutionize building design. You can give us the 'light look.' The sky's the limit! Don't fail us!"

Referring to the PCI's proposal for certification of prestressing plants, Mr. Luckman declared that a fundamental need is for an institute of research and standards. "Quality should come first," he said. "You have a good, universally recognized product. You need all kinds of assistance to your members. Develop your own seal of approval as a means of identification."

"We are at this moment in the middle of a marketing revolution which is developing two distinct sets of consequences. The first is the profound shifts taking place in the manufacturer's own function in the market place. As the most successful companies are proving today, the maker must be a good marketer if he doesn't want his competitor to move ahead of him in the market place. Thus, the manufacturer is changing from being maker-minded to being market-minded."

Emphasizing the importance of ideas in the area of mass marketing, Mr. Luckman asserted that the kind of ideas may have even more to do with success than the weight and numbers of dollars put behind the ideas. In this connection he offered a slogan which he said was especially suited for use by the prestressed concrete industry. This was: "Sell and stress. Sell the stressing and stress the selling."

"Building Today's Successful Marketing Program," was the title of the opening paper at the afternoon session, Sept. 21. The speaker was James R. Hawkinson, professor of Marketing, Northwestern University, Evanston, Ill. He said that most successful companies have been innovators, developers of new products and processes. "Sometimes problems of production take precedence over marketing," he said, "but there is increasing recognition of the importance of marketing. This is true for four reasons," Professor Hawkinson said. "(1) Increased competition. More people are after the consumer's dollar. (2) Increasing costs and slipping net profits. (3) Tested market practices are better known today. (4) You can now secure the services of skilled specialists who can show you how to sell a higher volume at less cost."

Professor Hawkinson said a series of coordinated activities are essential elements in a successful marketing program. "You must have a well-planned program. Determine the most productive segment of your market and go after it. Your marketing budget is your guide. This should be made on a quarterly basis. The sales force is generally the highest item of expense. Sales should be increased faster than any increase in salesmen's expenses."

Ralph E. Knight, vice president and director of research and development, Kaiser Aluminum & Chemical Corp., Oakland, Calif., followed Professor Hawkinson with a paper on the announced subject: "Key to the Future — Research and Product Development." Mr. Knight, who was introduced by Douglas P. Cone, Florida Prestressed Concrete Co., Tampa, suggested that the title of his paper should be changed to: "Research and Development — One Key to the Future."

"Prestressed concrete," Mr. Knight said, "is a unique material of construction which must compete with the better known and older materials." He referred to aluminum as an example of rapid market growth in uses as a result of a research and development program which he said plays an important role in any marketing effort. Such an activity, he said, involves improvement of product and development of new uses and processes. He declared that a successful marketing program must have a background of complete data and engineering facts about the product.

The concluding feature of the first day's program was a panel discussion on: "Market and Product Development, Research and Pricing Policies," which generated spirited and informative comments. Robert H. Singer, manager, Prestressed Concrete Division, Ben C. Gerwick, Inc., Napa, Calif., was panel moderator. Panel members were: Elmer Clark, vice president, Superior Sand & Gravel, Phoenix, Ariz.; Leonard M. Perlmutter, vice president, Prestressed Concrete of Colorado, Inc., Denver; William A. Haley, American-Marietta Co., Chicago; Alfred L. Parme, manager, Structural and Railways Bureau, Portland Cement Association, Chicago; John B. Smythe, assistant to the president, Atlantic Prestressed Concrete Co., Inc., New York, N. Y.

Moderator Singer announced that a series of pertinent questions would be thrown on the screen and answers and comments invited from the panel members. Some of the questions with briefs of answers and comments as they were given, follow:

QUESTION: What should we, as an industry, consider as the market for prestressed concrete?

ANSWER by Mr. Smythe: Schools, hospitals and other public buildings.

QUESTION: What do you consider basic fundamentals of a sound marketing program?

ANSWER by Mr. Dubois: You must know what you are going to produce and for what market.

QUESTION: What is your idea of a local market development program for prestressed concrete?

ANSWER by Mr. Clark: Personal contacts are most successful but brochures and mailing pieces should also be used.

QUESTION: While we all have our own development and research programs in varying degrees, most of us are not in a position to undertake national or major programs. What can be done to meet this problem?

ANSWER by Mr. Smythe: Technical literature is an urgent need.

COMMENT by Mr. Perlmutter: Literature to be effective must be made available to the right people.

COMMENT by Mr. Singer: There is still a hard job ahead in selling the basic concepts of prestressing.

QUESTION: In developing a market for our product we must consider the competition. Sometimes this is steel, sometimes reinforced concrete, sometimes timber. How can this competition best be met?

ANSWER by Mr. Perlmutter: We must show that our product is best for specific purposes.

COMMENT by Mr. Clark: Delivery time is more important than price.

COMMENT by Mr. Singer: We must be the competition rather than the competitor.

QUESTION: How can the prestressing industry coordinate marketing efforts to meet the aggressive campaign undertaken by steel companies to regain their former position in the bridge market?

ANSWER by Mr. Parme: Cost is an important factor. Refers to experience with prestressed girders for bridge on Illinois Toll Road.

COMMENT by Mr. Dubois: Points to the "light look" and other appearance advantages over steel.

QUESTION: Many agencies both private and public are requiring plant production by established companies for all precast, prestressed units. What is your opinion of this requirement?

ANSWER by Mr. Dubois: The question is what are established companies? I believe that potential users look to established companies.

QUESTION: What assistance do you gain in market development work from your suppliers such as cement companies and steel companies?

ANSWER by Mr. Smythe: We are getting a great deal of such support.

COMMENT by Mr. Perlmutter: We have had tremendous help from cement companies and from the PCA.

QUESTION: What can you do to be the low bidder?

ANSWER: This question generated a lively discussion from the floor as well as among panel members. Consensus was that price is not always the main issue.

QUESTION: What are some of the new products in prestressed concrete which can be developed and expanded during the next five years? How would you propose that this development be done?

ANSWER by Mr. Haley: Some producers are thinking of prestressed cribbing, for one thing. Also parking garages are a tremendous market. Uses of thin-shell sections are promising.

QUESTION: Have you discussed with your engineering or research staff, all important past, current and prospective technical developments which may affect your future position?

ANSWER by Mr. Perlmutter who says that as a result of such discussions he realizes the rate of obsolesence in the industry is terrific and that you must be flexible and able to roll with the punch. The manufacturer can't afford to relax for a moment. He must keep re-examining and redesigning as well as reinvesting.

QUESTION: How can findings of technological research be transmitted to sales?

ANSWER by Mr. Dubois: By education of architect, engineer, contractor and all concerned. Should use demonstrations and engineering tests. Refers to fact that prestressed concrete is not recognized in New York building code.

QUESTION: What do you consider to be a sound and fair pricing philosophy?

ANSWER by Mr. Smythe: We have to price ourselves competitively in our own market. You must base your price on a reserve for research.

COMMENT by Mr. Perlmutter: Fluctuations in price have almost stopped in our market. I believe prices will level off.

COMMENT by Mr. Singer: There is too much selling on price rather than on the advantages of prestressed concrete.

QUESTION: Do you approve of bid-shopping, price-cutting policies of some manufacturers?

ANSWER by Mr. Clark: No, because such practices are unethical and harmful to the industry.

A panel discussion on "Yard Handling, Transportation and Erection Techniques," opened the Tuesday morning program. Moderator was Peter J. Verna, Jr., president, Concrete Materials Co., Charlotte, N. C. Panel members were: E. H. Leffelholst, of Material Service Corp., Chicago; Leo J. Jasien, superintendent, prestressed concrete, American-Marietta Co., LaGrange, Ill.; W. J. Blanton, chief engineer, Concrete Structures, Inc., Richmond, Va.; D. O. McCall, chief engineer, Basalt Rock Co., Napa, Calif.; and Glenn R. Johnson, Jr., Materials Handling Institute, Washington, D. C. Members of the panel were asked to discuss their own plant and field experience.

Mr. Johnson, who said he had just completed visits to 160 prestressing plants, listed what he regarded as the five most important considerations in establishing a prestressing plant. These were: (1) Capital required. (2) Land available. (3) What to manufacture. (4) Means of quality control. (5) Storage area, accessibility to trucks and handling equipment.

There are three types of plants, Mr. Johnson said, which may be classified as: Small inland plants, large inland plants and water front plants. "Do not copy your neighbor's plant," he advised. "Your problems may be entirely different from his." Mr. Johnson suggested that climate influences the type of plant and the means for quality control. He warned that provisions for storage may be costly under some conditions.

Moderator Verna, in summarizing the discussions, said it was the consensus of the panel members that good housekeeping in a plant means dollars. A profitable plant, he said, is one in which advance study has been given to internal movements. He suggested that a philosophy of material handling should show that the best system is the use of what you have. He said he believes that each producer should and must maintain control of his product from plant to the finished structure.

Appearance of finished prestressed concrete members is almost as important as quality, according to members of a panel on "Quality Concrete for Prestressing," the second feature on the Tuesday morning program. Joe Schneider, chief engineer of the Edward Campbell Co., Vineland, N. J., was Moderator. Panel members included: Dan L. Sutter, Anchor Concrete Products, Inc., Buffalo, N. Y.; Rudolph C. Valore, Jr., Texas Industries, Inc., Dallas; Robert Lyman, chief engineer, Atlas Structural Concrete, Inc., El Paso, Texas; and Harry Colthurst, of Ryan Builders Supplies, Ltd., Windsor, Ont.

Panel members agreed that a quality control laboratory at the plant is of the greatest value. Various members of the panel referred to the influence of weather variations on the quality of concrete; described methods of curing and reported on the strengths obtained at various ages of the concrete. Mr. Lyman emphasized the importance of uniformity in a concrete mix. He also referred to the difficulty of curing concrete in hot, arid regions such as the southwestern part of the United States and said curing under such conditions affects ultimate strength and increases the tendency toward cracking.

Mr. Colthurst said the Ryan plants in Windsor and London, Ont., experienced no difficulty in producing quality concrete in cold weather. Mr. Valore asserted that it is possible to predetermine qualities of concrete, especially low shrinkage, low creep and modulus of elasticity. "We believe in an intensive degree of quality control in accordance with ACI standards," he said. Mr. Valore also reported that Texas Industries has a full time man on quality control. Hourly checks are made on lightweight concrete. He said that

VOLUME TIGHTLY BOUND

producing 5,000 psi concrete at 28 days presents no problem and suggested that producers should aim at 4,000 psi in 18 to 20 hours. He said Texas Industries plants are able to get strength of 9,000 psi in 28 days.

The panel also discussed the specific question: "How Does Quality Affect Camber?" Mr. Schneider said that to control camber, producers should design high strength concrete and release forms at the same time each day.

Opinions varied on the question of admixtures. Several producers said they used no admixtures. Mr. Valore expressed the opinion that most admixtures do not help workability; that they are over-rated and are of value chiefly as retarders and should not be used without first making a trial batch. Mr. Colthurst said that in his experience, portland cement is the best admixture.

The draped strand method of pretensioning will be used more and more in future prestressed concrete, in the opinion of panelists opening the Tuesday afternoon program. The discussion of "Draped Strands at the Illinois Toll Road and Elsewhere," generated a lively interest. M. E. Bender, of Knoerle, Graef, Bender and Associates, Inc., Baltimore consulting engineers, was Moderator. Mr. Bender was chief bridge engineer of the consulting firm during the construction of 224 prestressed concrete bridges on the Illinois Toll Highway. Members of the panel were: Philip E. Balcomb, American-Marietta Co., La Grange, Ill.; Charles H. Goodman, Material Service Corp., Chicago; Wm. J. Talbot, Jr., chief engineer, Precast Concrete division, Ben C. Gerwick, Inc., San Francisco; A. H. Gustaferro, Consumers Company division, Vulcan Materials Co., Chicago; and Jacob O. Whitlock, president, Midwest Prestressed Concrete Co., Rochelle, Ill.

Several members of the panel described the tensioning procedure in their plants. Mr. Whitlock told of tensioning four or five strands at the same time, using three deflection points. Mr. Talbot said the procedure in the Gerwick plant involves use of special equipment developed in the plant. He said strands are stressed in two separate groups before deflecting is done. Fixed horizontal screws are used at hold-down points.

Mr. Gustaferro told of casting up to nine girders at a time on four beds, 240 to 440 ft. long. Each strand is stressed separately. Strands are repositioned at the time of tensioning. In longer beds the strands are jacked from first one end and then the other. Mr. Gustaferro said he believed future developments would call for draped strands.

Mr. Balcomb described stressing operations at the Hodgkins, Ill. plant of the American-Marietta Co., where he said strands are stressed from both ends simultaneously. He said it should be left to the designer to decide whether to adopt the draped strand method but that the question should be given good enginering scrutiny.

A symposium on "Interpretation of Specifications in Relation to Inspection," was the final feature on the Tuesday program. Moderator was Jack R. Janney, Chicago consulting engineer who designed the test bridge and supervised the tests which influenced the adoption of prestressed concrete for the Illinois Toll Highway bridges. Panel members were Ole Larsen, materials engineer, Illinois Division of Highways; Al Parme, manager Structural & Railways bureau, Portland Cement Association, Chicago; E. L. Peck, plant manager, Midwest Prestressed Concrete Co., Springfield, Ill.; J. J. Waddell, chief materials engineer, Knoerle, Graef, Bender and Associates, Baltimore; and E. L. Erickson, chief of the bridge branch, U. S. Bureau of Public Roads.

Mr. Erickson spoke at some length on the need for specifications. "Buyers must be assured of a durable and safe product when it is used on any structure," Mr. Erickson said. "Engineers must have guides to control design and construction. Draped strands present only one of many questions. For one thing, sharp bends in strands should be avoided. Specifications do two important things," Mr. Erickson said; "Give engineers a basis for design, and protect owners from faulty designs. Inspection is a vital part of engineering construction," the speaker declared. "If prestressing is to continue its success, careful inspection must be maintained."

Following Mr. Erickson's remarks the panel began discussion of a number of related questions: (1) Why should material specifications for prestressed concrete be different from specifications for cast-in-place structural concrete? (2) Relationship between inspectors and producers? (3) What should be the qualifications for inspectors?

On the first question, panel members agreed that there should be new and different specifications for prestressed concrete in view of the higher strengths required. It was also suggested that there should be inspection of aggregates at the source. In the matter of relations between producers and inspectors and the qualifications of inspectors, panel members agreed that a background of engineering experience is desirable; that there would be a clear understanding between producer and inspector as to what each should do; that both should work together as a dedicated team; that inspectors should not deal directly with workmen on the job; that the producer should understand that the inspector's not responsible for errors; that neither should expect or receive favors. It was agreed that careful specifications should be worked out to avoid damage in lifting, handling and transportation of prestressed concrete units.

"Prestressed Highway Pavement's Future Potential," was the subject of a paper which opened the morning session on Wednesday. The speaker was John J. Murray, development engineer of Jones & Laughlin Steel Corp., Pittsburgh. Mr. Murray predicted that because of the magnitude of the national highway program, the use of prestressing in highway pavement can conceivably double the use of the general method of prestressing concrete. Mr. Murray designed and built an experimental prestressed concrete pavement for Jones & Laughlin. He said the project had the distinction of being the first full-sized prestressed concrete highway pavement constructed in America. The prestressing steel consisted of 24 strands, 7/16-in. in diameter enclosed in six 1½-in. diameter flexible steel ducts. The two 200-ft. sections of the main pavement were left with a gap between them. The required prestress was obtained by jacking these two sections apart a distance of 2 ft. Details of construction and all mechanical features were shown by Mr. Murray in a series of slide pictures.

Following Mr. Murray's paper, attention of the meeting was concentrated on the general theme, "The Challenge of Marketing;" Jackson Taylor, senior vice president of Lennen & Newell, Inc., New York advertising agency, spoke on "Advertising's Place in a Marketing Program." To emphasize his belief that outside of the construction industry, prestressed concrete is practically unknown, Mr. Taylor said that after accepting the invitation to address the Prestressed Concrete convention, he mentioned the product at different times to 35 men all holding positions on a policy-making level in important industries. Not one of these men had even heard of prestressed concrete.

"There are two things that impress me mightily about your product," he said. "These are the tremendous increase in all types of construction which will undoubtedly take place during the next twenty years, and the advanced nature of your product itself as a building material." He emphasized the fact that the construction industry is first in dollar volume among all American industries and is also among the healthiest of American industries. In this connection he quoted a prediction of one economist to the effect that during the coming decade, new construction will just about equal the present depreciated value of all private structures, homes, office buildings, factories and stores now in the United States.

Mr. Taylor then suggested that these tremendous needs cannot be satisfied with the present-day building materials. "For the past few years there has been an evolutionary change taking place in the nature of building materials in the United States," he said, "From the simple, natural traditional materials, toward more complex materials; from almost complete dependence on compressive strengths and great weight toward lighter structures which enjoy the strengths of new materials in tension. At the same time there has been a development in the direction of building structures from parts, rather than from pieces. Within both of these developing trends are many of the answers to the social and economic problems with which the construction industry will be faced during the next decade. And within these trends also lie possibilities for fame and fortune without measure for architects, engineers, contractors, manufacturers and others in and about the construction industry."

A panel discussion on "Advertising, Sales and Promotion," followed Mr. Taylor's address. Norman L. Scott of R. H. Wright & Son, Fort Lauderdale, Fla. was Moderator. Panel members were: Jack Streblow, sales manager, Structural Concrete Products division,

Basalt Rock Co., Napa, Calif.; R. O. Kasten, Union Wire Rope Corp., Kansas City; Dale Delano of Thomas Concrete Pipe Co., Oklahoma City; Raymond J. Gross of the Formigli Corp., Philadelphia; and John K. Zimmerman, vice president, Pennsylvania Prestress, Inc., York, Pa. Questions for discussion by the panel were thrown on the screen. These questions and the answers and comments by panel members included the following:

QUESTION: What is best method of getting architects and engineers to accept and use prestressed concrete? Is approach different for buildings than for bridges?

ANSWER by Mr. Delano: Believes demonstrations are effective and that emphasis should be placed on durability, economy and availability, especially on fact that there is no shortage of materials used in prestressed concrete. Mr. Delano also suggested a guarantee of price before contract is let.

COMMENT by Mr. Streblow who says that talks at professional meetings are effective promotion and time well-spent.

QUESTION: Do you feel that more effort should be directed toward architects rather than structural engineers?

ANSWER by Mr. Streblow: Architects should get more attention where they are concerned with the shape and form of a structure.

QUESTION: How to get prestressed concrete specified for a maximum number of bridges in the Federal highway program?

ANSWER by Mr. Zimmerman: Should work closely with your state highway officials. Be honest in your pricing and engineering approach.

QUESTION: Of what value to your sales and promotion is the work of state associations of prestress manufacturers?

ANSWER by Mr. Zimmerman: The help of such associations is of great value especially on public projects and in working with state highway departments. An association representative can get a hearing in meetings more readily than the representative of a single manufacturer. There are now state associations in Illinois, California, Florida and Pennsylvania.

QUESTION: Do architects and engineers have sufficient design data in written form?

ANSWER by Mr. Streblow: Not yet but someone must see to it.

COMMENT by Moderator Scott: Local representatives of the manufacturer should see that such data is provided.

QUESTION: Should a salesman of prestressed products be an engineer?

ANSWER by Mr. Zimmerman: Such a salesman should certainly have a basic understanding of engineering.

COMMENT by Mr. Gross: I believe any engineer can be a salesman and that salesmen can be given engineering training. Salesmen should know more about the product than the customer knows.

QUESTION: Should advertising be addressed to the general public?

ANSWER by Mr. Gross: Yes, you can address the general public with institutional advertising.

QUESTION: What are prestressing steel manufacturers doing in advertising prestressed concrete?

ANSWER by Mr. Kasten: The steel companies have not been selfish. They are helping to create a market. Case histories based on specific projects have been used in steel company advertising. I'm sure that strand producers plan to continue to run job stories in their advertising and to continue to work with highway engineers as they have been doing.

The first event on the Thursday morning program, with A. H. Gustaferro as Chairman, was a report by J. Ashton Gray, chairman of the PCI Plant Standards committee. This related to the proposed certification of prestressing plants which meet certain standards of plant, equipment and materials. The report sparked a lively discussion led by Mr.

Gray who declared there is an acute need for a definition of minimum standards for prestressing plants. The Plant Standards committee was appointed a year ago. Now, Mr. Gray said, the PCI directors want members to approve a plan to employ a competent inspection and regulating agency and back the plan with national advertising. With 260 producers now operating in the United States, Mr. Gray said a self-regulation and quality verification program are urgently needed. PCI members are to be canvassed on their opinions as to cost and means of financing the proposed program.

Members were told that possible cost for the verification program might run from $1,500 to $1,700 per plant and that if advertising were approved the overall annual cost for each plant might be as much as $3,500. There was a spirited discussion on the floor as to whether the proposed program could be financed on a fixed fee basis or a sliding scale rate based on annual tonnage of each plant. Means of enforcement of proposed minimum standards were also discussed.

The second feature of the Thursday morning program was a panel discussion on "Longer Span Prestressed Bridges." This was led by Morris Schupack of Schupack & Zollman, consulting engineers, New York City and Newton Square, Pa. Members of the panel were: Felix Kulka of T. Y. Lin & Associates, consulting engineers, Los Angeles; Russell Fries, construction engineer, DeLeuw, Cather & Brill, Chicago; Andrew J. Brown, of Skidmore, Owings & Merrill, Chicago; Noel Willis, bridge engineer, department of streets, City of Philadelphia; and Louis P. Saenz, consulting engineer, Havana, Cuba.

Mr. Schupack showed slides of details and finished views of many notable prestressed concrete bridges. Among these were the Walnut Lane and Gorgas Lane bridges in Philadelphia and the Oneida Lake bridge near Syracuse, N. Y. Other bridges shown and discussed by panel members included the Windsor bridge at Hartford Conn., Feather River bridge in California and the Montrose Avenue bridge in Chicago. Mr. Schupack told members of the Institute that he believed lightweight aggregate may be used in prestressed bridges before long.

Professor Saenz described a number of notable, long-span prestressed bridges built in Cuba. He said several such bridges with spans from 350 to 900 ft. long are being planned and asserted that spans up to 2,000 ft. long can be designed.

The last feature of the Thursday morning session was a paper by Edwin N. Searl of the National Fire Protection Association, on "Fire Ratings and Building Codes." Mr. Searl is Chairman of the Association's committee on Construction of Buildings and is superintendent of engineering, Western Actuarial bureau.

"Evaluation of fire experience is not an exact science," Mr. Searl said. "Many variables affect fires. It is important to develop information under test conditions. Fire insurance rates are based on degree of hazard and extent of protection. Safety is the purpose of building codes. Modern codes are based on performance standards. Your industry is facing up to its responsibility. I believe you will come up with the right answers."

The closing session of the convention Thursday afternoon began with a report on the Berlin prestressed concrete conference in the summer of 1958, by Ben C. Gerwick, Jr., retiring PCI president. He said there were 1200 delegates from 40 different countries. Thirty PCI members attended. Berlin reconstruction, Mr. Gerwick said, is being accomplished largely with prestressed concrete.

Professor T. Y. Lin of the University of California, Berkeley, whose work is generally recognized as having speeded up the development of prestressing in the United States, spoke on "Prestressed Concrete in the U.S.S.R. Relative to U.S.A. Practice." Professor Lin said the Russians are ahead of us in certain areas and behind in others. In the field of production, he said the Russians have advanced standards of mechanization. One point of difference between American and Russian standards he noted was the fact that their construction is not too attractive.

Philip E. Balcomb of American-Marietta Co., in closing the final program, reminded the PCI members that: "Your industry has grown up. It has gained national recognition and assumed responsibilities. It is now generally recognized that there is a prestressed concrete industry." He announced that the 1959 convention would be held at the Deauville Hotel in Miami Beach, November 1 to 7.

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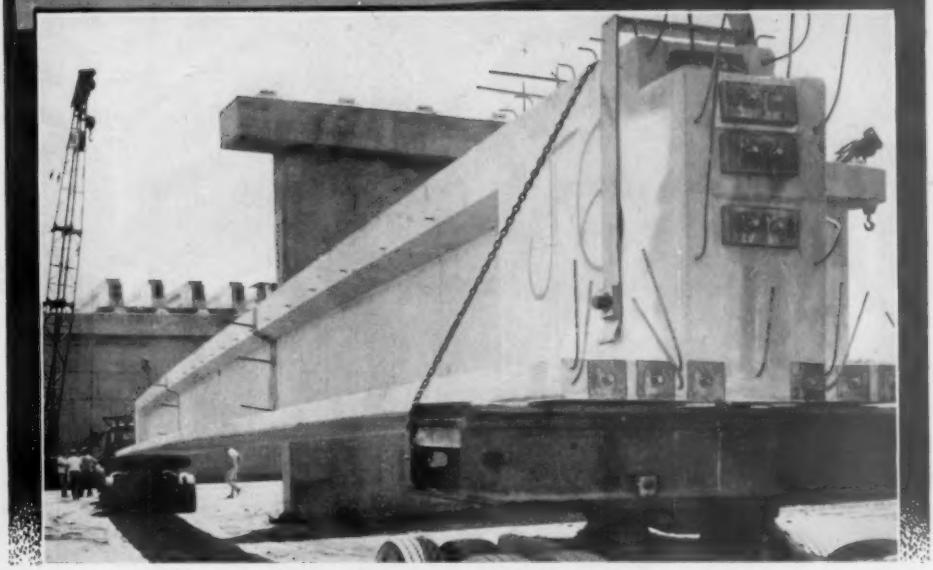

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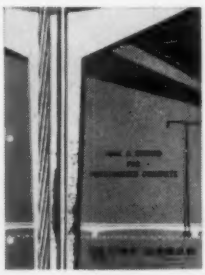

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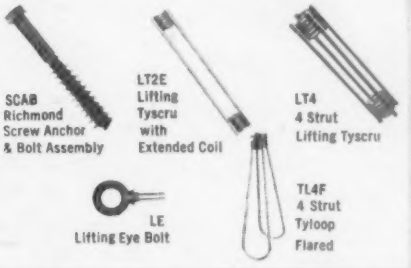

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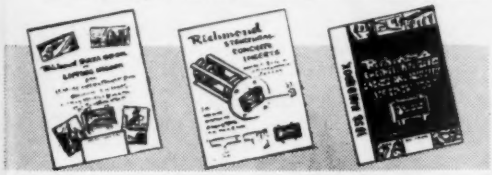

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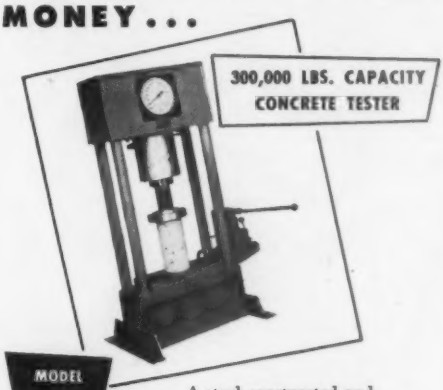

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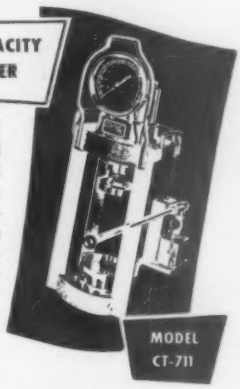

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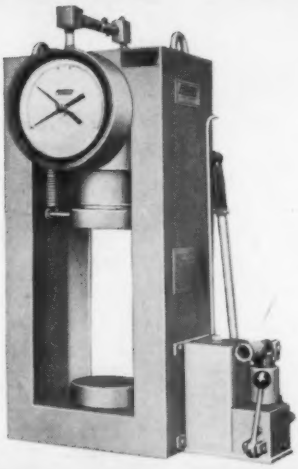

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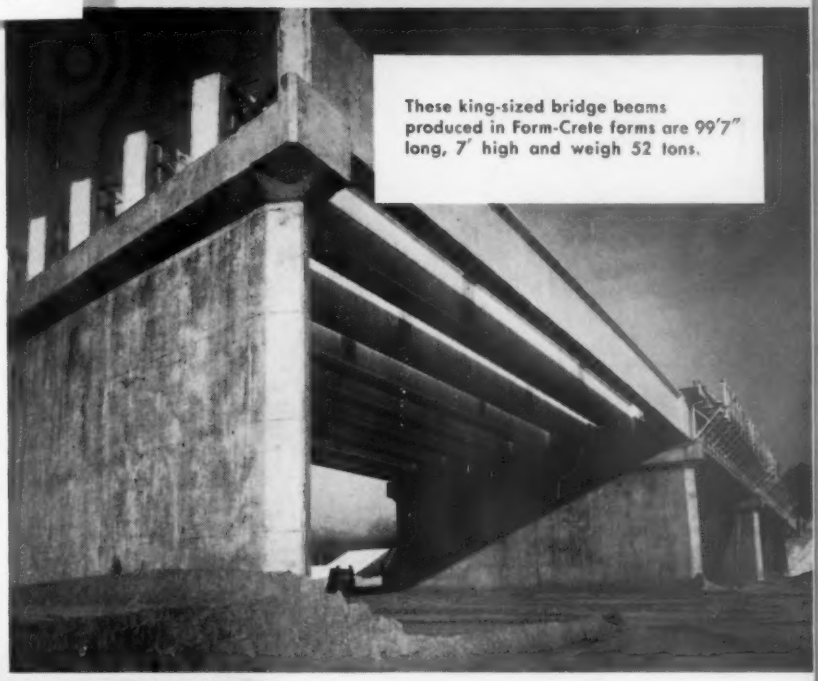

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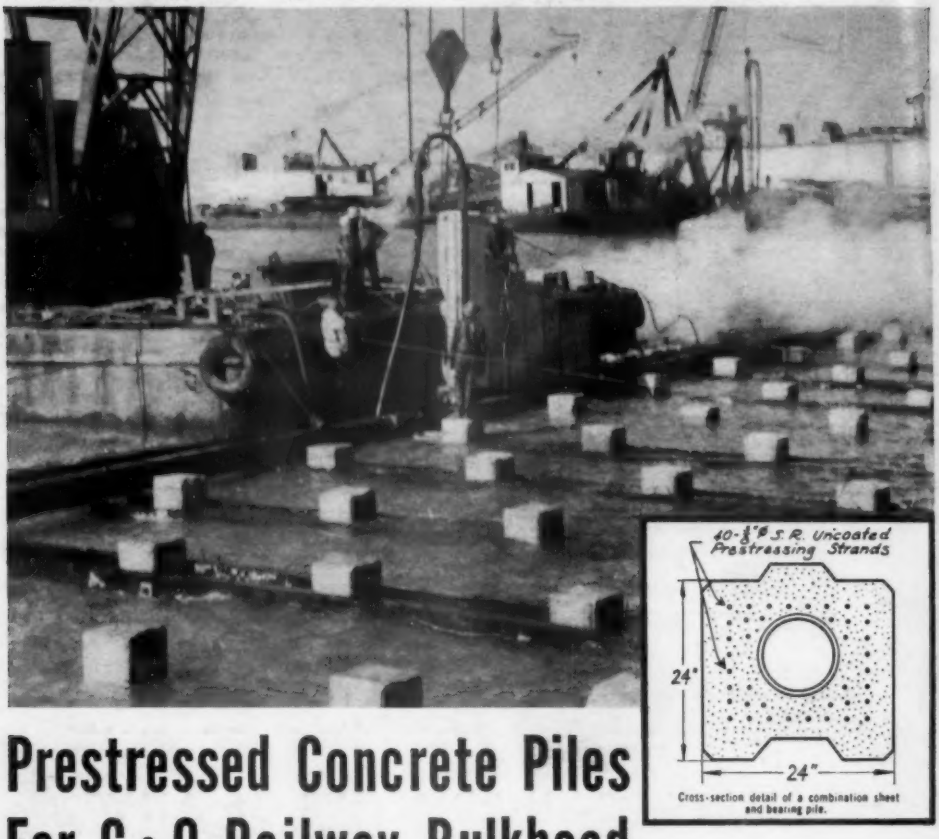

Cross-section detail of a combination sheet and bearing pile.

Prestressed Concrete Piles For C & O Railway Bulkhead

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